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CONSOLIDATION OF RECENT ALLUVIAL DEPOSITS IN BRITISH COLUMBIA

by

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled CONSOLIDATION OF RECENT ALLUVIAL DEPOSITS IN BRITISH COLUMBIA submitted by CARL FRANKLIN HUNTER in partial fulfilment of the requirements for the degree of MASTER OF SCIENCE.

ABSTRACT

Many of the valleys of British Columbia are infilled with several hundreds of feet of alluvial silt-clay deposits in a loose-density state. Each year more of these deposits are being used as foundations for major structures such as the Duncan Lake dam, Mission dam (now the Terzaghi dam), and the Kitimat Smelter. The settlements due to these structures have in every case exceeded the values computed by conventional settlement analysis based on one-dimensional consolidation theory.

The anomalous settlement behavior of the Duncan Lake dam as compared to conventional settlement analysis has been investigated. An attempt was made to determine the reasons for the variation between actual and computed settlements. One-dimensional consolidation and anisotropic triaxial consolidation tests were performed on sensitive silt-clays from Duncan Lake and Fort Francis, Ontario to provide a basis for analysis of field settlement data.

Based on data from piezometers, surface settlement gauges, and deep seated magnetic gauges, secondary compression was found to account for much of the total settlement at the Duncan Lake dam. Field C_c values were found to be up to 3 times larger than laboratory test results indicated. The difference in field and laboratory C_c values has been attributed to the effect of sample disturbance and the increase in the rate of secondary compression with pressure.

The increase of C_c values with load computed from field data has been attributed to the increase in the rate of secondary compression with pressure. The laboratory testing program showed that secondary compression increases to some maximum value and then decreases with total pressure; however, the rates of secondary compression in the laboratory tests were several times lower than the field rates at similar total pressures.

The anisotropic triaxial consolidation tests produced rates of secondary compression about twice those found in the conventional one-dimensional tests. As the stress environment, especially with regard to allowing lateral strain, can be controlled in the triaxial cell this test method appears to offer the greatest promise to adequate settlement analysis. Based on thin-wall Shelby tube soil samples conventional one-dimensional tests are shown not to be adequate for settlement analysis of structures located on sensitive loose silt-clay.

A hypothesis is presented which states that a field e-log p curve can be constructed by adding accumulated amounts of secondary compression to a basic e-log p curve due to primary consolidation. Application of the hypothesis is limited by the difficulty in obtaining accurate field rates of secondary compression.

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GLOSSARY OF TERMS

COEFFICIENT OF SECONDARY COMPRESSION..C_α (dimensionless)...The rate of change of void ratio with respect to the log of time, with zero time being taken as the time of instantaneous application of load. In the case of a gradually increasing load increment, average time over which loading occurs is taken as zero time. For the case of linear void ratio change with the log of time, C_α equals the change in void ratio over one log cycle of time.

COMPRESSIVE INDEX ..C_c..(dimensionless)...The rate of change of void ratio with respect to the log of pressure. If the plot of void ratio change with pressure is linear C_c equals the change in void ratio over one log cycle of pressure.

LIQUIDITY INDEX ...

$$\text{Liquidity index} = \frac{\text{(natural soil moisture content)} - \text{(plastic limit)}}{\text{plastic index}}$$

The liquidity index has a value of 1.0 for a clay with a natural moisture content equal to the liquid limit.

SENSITIVITY ... of a cohesive soil...

$$\text{sensitivity} = \frac{\text{unconfined compressive strength undisturbed}}{\text{unconfined compressive strength remoulded}}$$

The values for most clays range between 2 and 4, for sensitive clays between 4 and 8, and for highly sensitive clays greater than 8.

SENSITIVITY ... of a cohesionless soil ... Sensitivity of a cohesionless soil arises from a loose soil structure as defined within the context of relative density.

SILT-CLAY ... A soil with grain sizes predominantly less than 0.06 mm in diameter and containing appreciable proportions of both silt and clay sizes.

CHAPTER I

INTRODUCTION

1.1 General

Conventional settlement analyses are based on the Terzaghi theory of one dimensional consolidation. The accuracy of the results are therefore dependent on the applicability of the theory to the field conditions. Terzaghi recognized that his theory did not always accurately predict field settlements but he was unable to mathematically account for the difference.

The settlements resulting from prolonged effect of "secondary compression" have been of particular concern in British Columbia where evidence is accumulating that secondary compression may exceed primary consolidation by several times during the life of a structure.

Much of the most valuable land in the province is located on valley floors underlaid by up to several hundreds of feet of normally consolidated sands and silt-clay layers.

The Duncan Lake dam, which is one of the three Columbia River projects now under construction in British Columbia, is being built across a valley in which there are about 400 feet of loose-density alluvial deposits. Settlement and piezometric readings acquired at this site during construction of the dam have provided the most extensive data ever available to check the performance of such natural deposits against the consolidation theory.

1.2 Purpose of the Investigation

This study attempts to correlate the settlement performance of the Duncan Lake dam with the theory of primary and secondary consolidation and to explain some factors influencing the rate and amount of secondary compression in sensitive silt-clays.

1.3 Scope of the Study

The field data used in this report are from the Duncan Lake dam. Some reference is made to settlement records in other areas of British Columbia, however these records have been used only to emphasize the significance and extent of the problem.

Field settlement data from the Duncan Lake dam cover the period from May 1965 when construction was started, to March 1967 as this thesis was written. Completion of the Duncan Lake project is expected by June of 1967. The settlement data consist of readings from special magnetic gauges set deep into the silt-clay deposits and from surface settlement gauges. Piezometer readings, three dimensional alignment data and construction sequence information comprise the remainder of the available field data.

A modest laboratory testing program was set up to determine the influence of shearing stresses on consolidation characteristics of normally consolidated silt-clays. Samples were tested from both the dam site and from the Fort Francis area of Ontario. Both soils used were moderately sensitive. The testing consisted of one-dimensional consolidation tests and anisotropic consolidation tests in triaxial cells.

CHAPTER II

THEORETICAL AND PRACTICAL ASPECTS

2.1 General

The theoretical explanation for many of the prolonged foundation settlements recorded at various places in British Columbia appears to be inadequate. The settlements are frequently of such magnitudes that from a practical point of view they present serious design problems.

Many of the valleys and fjords of British Columbia are filled with great depths of normally consolidated, fluvio-glacial, lacustrine and alluvial deposits. With these deposits field settlements are being recorded that differ significantly from what is to be expected from the basic Terzaghi theory of soil consolidation.

This chapter outlines the current theories of soil consolidation pertinent to the problem. The concluding sections briefly describe three case histories which emphasize the need for more precise knowledge of soil consolidation.

2.2 Review of Pertinent Theory

Consolidation of silt and clays under an applied load or loads is an extremely complex process. Until a microscopic knowledge exists of individual particle contacts, orientation, elastic properties, and adsorbed water hulls, no theory of

consolidation is likely to be more than empirical. That is to say, even the most complex theories describing consolidation must be based on visible manifestations of microstructure behavior. Some present day theories consist of formulae which define quite well the behavior of cohesive soils in compression. The theories are however extremely complex. Regrettably the amount of simplification which can be used to make these complex equations more applicable to field conditions is limited because of inadequate knowledge of field behavior and physical chemical bonding between soil particles.

The mechanics of primary consolidation are based on the premise that for any void ratio existing in a soil skeleton there exists a maximum effective stress which can be supported without deformation. When a pressure is applied in excess of the soil structure capacity the excess pressure must be supported by the pore water. The developed pore water pressure causes a flow of water from the soil. The void ratio decrease resulting from the flow of water continues until pressure induced by the added load is entirely supported by the soil skeleton at some lower void ratio. Based on this description the entire process of volume change can be expressed mathematically.

These expressions were first derived by Terzaghi for a condition of one-dimensional flow and volume change. For a laterally confined sample the amount of volume change ΔV, is proportional to the change in sample height ΔH,

$$\frac{\Delta H}{H_o} = \frac{\Delta V}{V_o} \qquad 2\text{-}1$$

Similarly, the relationship between void ratio, e_o, and the sample height, H_o, can be expressed as follows:

$$\Delta H = H_o \frac{\Delta e}{1 + e_o} \qquad 2\text{-}2$$

By the conventional testing in a one-dimensional compression test a relationship between void ratio and effective stress can be determined. The slope of the curve expressing the relationship is called the coefficient of compressibility a_v .

$$a_v = - \frac{de}{d\sigma'} \qquad 2\text{-}3$$

The value of a_v decreases with increased effective pressure σ' . As a_v continually varies with σ' a more convenient relationship between volume change and the soil characteristics is found by plotting e versus log σ'. The slope of this curve is called the compression index C_{c} ,

$$C_{c} = \frac{de}{d(\log_{10} \sigma')} \qquad 2\text{-}4$$

It can therefore be shown that ΔH can be expressed in terms of C_{c} as the soil characteristic

$$\Delta H = H \frac{C_{c}}{1 + e_o} \operatorname{Log}_{10} \frac{\sigma'_o + \Delta\sigma'}{\sigma_o'} \qquad 2\text{-}5$$

where σ_o' and $\Delta\sigma'$ are the average effective stresses acting on the mid-point of the soil depth H .

The relationships shown above are all that are necessary to

compute ultimate compressions in one-dimensional consolidation. The accuracy of settlement analyses based on one-dimensional compression therefore depends on the accuracy with which the variables in equation 2-5 can be assessed.

Except for highly sensitive soils the compression index C_c has been found to be approximately a straight line on a plot of void ratio vs. log of effective stress for pressures exceeding the previous maximum loading on the soil (Taylor, 1948). The empirical equation,

$$e = e_o - C_c \log \frac{\sigma'}{\sigma'_o} \qquad 2\text{-}6$$

describes this relationship. For purposes of settlement computation the C_c value for normally consolidated soils is therefore essentially constant.

The numerical value of C_c can be computed in several ways. The conventional method is to develop an e-log p curve based on a one-dimensional consolidation test. The value of C_c can be taken directly from the straight line portion of the graph. A second method is empirical and based on the relationship between the liquid limit and C_c' (Terzaghi and Peck, 1948), where C_c' is the compressive index of the soil in a remoulded state. For medium to low sensitive clays the following approximate relationship exists:

$$C_c \approx 1.30\, C_c' = 0.009\,(Lw - 10\%) \qquad 2\text{-}7$$

The third method of determining C_c is based on the results of settlement field data. Equation 2-5 can be put in the form:

$$C_c = \frac{\Delta H}{H} \cdot \frac{(1 + e_o)}{\log \left(\frac{\sigma_o' + \Delta\sigma'}{\sigma_o'} \right)} \qquad 2\text{-}8$$

Therefore the average C_c value for a given depth of soil H, can be found. The initial void ratio e_o can be computed from the simple relationship for saturated soils,

$$e_o = w \cdot G_s \qquad 2\text{-}9$$

where w is the initial moisture content and G_s the average specific gravity of the soil solids.

Limitations such as sample disturbance and test procedures of the first two methods for computing C_c are well known (Terzaghi and Peck, 1948; Taylor, 1948; Leonards, 1962). When field settlement data are available computation of C_c values becomes independent of sampling techniques and sample disturbance, but they may be subject to appreciable errors due to inaccuracies in H, and variations in soil types. Based on equation 2-7, C_c values can be computed from surface settlement gauge data, the depth of compressible soil, and the subsurface stress distribution due to the surface loading. Determination of the effective depth of stress influence is difficult. All existing theories of elastic stress distribution such as those of Boussinesq and Westergaard, predict a stress decrease approximately proportional to the square of the depth; therefore some stress exists even at an infinite depth. For simplification, the effective depth may be arbitrarily taken as the depth at which the stress is 10 percent of the surface stress. The fact that the effective depth is not definitely known and in fact varies with the method of stress analysis, places a serious limitation on the accuracy of the C_c value computed. Furthermore the distribution

of the stress beneath the point of load application is not usually linear, (Taylor, 1948; Leonards, 1962) and therefore simple averaging of the upper and lower stress values is accurate for only short distances. The accuracy can be imporved by averaging the stress to a theoretical stress distribution curve.

A settlement analysis of the soil in the field requires some knowledge of the depth to which settlements extend and the variation of C_c with depth and soil type. At the site of the Duncan Lake dam the settlement behavior with depth was obtained by installation of deep seated settlement markers. These gave the variation in settlement with depth, and permitted more accurate values of C_c to be computed for the soil between sets of consecutive markers.

The second aspect of one-dimensional consolidation theory is the rate at which consolidation proceeds. The mathematics for expression of the time relationship to volume changes are based on hydrodynamic principles. The resulting expressions themselves are not simple but can be found in most texts on soil mechanics, (Taylor, 948 and others).

The resulting general expression for void ratio change, Δe, with time t, is given by Wahls (1962) as:

$$\Delta e = a_v \Delta p \left[1 - \frac{8}{\pi^2} \sum_{n=0}^{\infty} \frac{1}{(2n+1)^2} \epsilon^{-\left[(2n+1)\frac{\pi}{2}\right]^2 \cdot C_v \cdot \frac{t}{H^2}} \right] \qquad 2\text{-}10$$

in which a_v is the coefficient of compressibility $= \frac{\Delta e_{100}}{\Delta p}$, C_v is the coefficient of consolidation, Δp is the change in effective stress, t represents time, H is the maximum distance to a drainage surface, and ϵ is the base of natural logarithms.

FIGURE 2-1* shows the theoretical time curve based on equation 2-10. For large values of t, the curve becomes asymptotic to some horizontal line.

In fact, laboratory test data and field settlement curves differ from the theoretical time curve. FIGURE 2-1 shows two typical laboratory or field curves, type I (FIGURE 2-1(a)) and type II (FIGURE 2-1(b)). The type I curve (Wahls, 1962) shows a close relationship to the theoretical curves up to the point of 70 to 80 percent consolidation. Beyond this range secondary compression effects become predominant and the volume change can no longer be expressed by hydrodynamic theory.

The coefficient of secondary compression C_α is expressed as the void ratio change over one logarithmic cycle of time. The slope of the secondary branch is usually a straight line on a semi-logarithmic plot, however there are some exceptions to this characteristic (Io, 1961; Leonards and Ramiah, 1959). To date no adequate mechanism for expression of the process of secondary compression exists. Indeed, there are still conflicting evidence and opinions about the physical characteristics of the process. Some aspects of the characteristics of secondary compression are presented in the following paragraphs.

The coefficient of secondary compression varies according to the soil type and the stress conditions. Most inorganic clays have a

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* All major figures are located in numerical order in Appendix A.

primary consolidation much greater than the secondary when compared after extended periods of time. The type I curve is typical of the above comparison. Other soils such as organic soils, and some normally consolidated silts exhibit secondary compression well in excess of the primary even after relatively short periods of time. The characteristic feature of the latter soil types is the absence of an inflection point as shown in the type II curve. As the point of inflection conventionally indicates the end of primary consolidation ie: zero pore pressure, no graphical method is available to determine the relative magnitudes of primary and secondary compression.

The first attempt at a quantitative approach to secondary compression was by Buisman (1936). He based his emperical formula on the reasonably accurate assumption that secondary compression is linear on a deflection versus log time plot. From consolidation tests on peat and clay Buisman made the following conclusions:

"1. The semi-logarithmic diagrams, with an exception for the vicinity of the loading point, are rectilinear as long as observations were continued.

2. Observed peat samples of 2 cm thickness produce rectilinear diagrams about 1 minute or a few minutes after loading and clay samples about 1 day after loading.

3. Diagram slopes (C_s values) are approximately proportional to the loads applied, with perhaps some tendency towards decreasing increments of slope for equal increments of load.

4. Superposition of loads, successively applied, results in superposition of log t $-\Delta H$ diagrams.

5. Higher temperature causes increasing slope of the diagrams."

The disadvantage of Buisman's empirical relationship is that linear relationships do not always exist (Gibson and Lo, 1961) and that C_α in a laboratory test may not equal the field value.

A rational theory to include both primary and secondary compression was published by Taylor and Merchant in 1940. The theory assumed that secondary compression began when primary consolidation terminated and the rate of secondary compression developed at a rate proportional to the undeveloped secondary. The chief objection to the Taylor Merchant theory besides its complexity is that secondary compression is concurrent with primary consolidation as well as subsequent.

Gibson and Lo (1961) published a paper which presented analyzed settlement data from a few field sources and numerous laboratory tests. Plotting of these results on deflection versus log of time showed three curve types. One curve consisted of a gradually reducing C_α value. The second type was most common and showed a linear relationship until secondary compression terminated. The third type curve consisted of an initially increasing rate of compression, then a fairly rapid reduction to some terminating void ratio. Generally speaking remoulded soil samples were found to fall into the first category.

Lo (1961) developed a theory based on visco-elastic concepts and derived equations to express the relationships produced by the three curve types. Lo's equations are extremely complex.

Further analysis by Christie (1964) showed that Taylor and Merchant's equations were identical to Lo's.

One of the broadest approaches to primary and secondary volume changes was done by Wahls (1962). Wahls developed expressions for the various factors which he found influenced primary and secondary compression. Probably more important than the expressions derived were the factors found which influenced consolidation. Wahls concluded from his test results on a calcareous, organic silt and from a survey of recent publications the following points:

"1. The coefficient of secondary compression, C_α , is dependent on the void ratio (and consequently the total pressure) and is independent of the magnitude of the pressure increment and the pressure increment ratio.

2. C_α represents the maximum rate, with respect to the logarithm of time, at which secondary compression occurs during a pressure increment. During primary consolidation secondary compression is occurring at a rate that is not linear with respect to time.

3. The time required for the rate of secondary compression to approach C_α is directly related to the time required for completion of primary consolidation.

4. As the pressure-increment ratio is reduced, the shape of the compression-logarithm of time curve differs more radically from the theoretical curve of primary consolidation. For sufficiently small pressure increment ratios ($\frac{\Delta P}{P_o} < 1/3$) the time at which primary

consolidation is completed cannot be determined by the conventional Casagrande method. In such instances the maximum slope of the curve during primary consolidation is less than the final secondary slope, thus there is no inflection point on the curve."

Leonards and Altschaeffl (1964) published a comprehensive analysis of secondary compression and on the indefinitely related subject creep. Some laboratory results were also published. The tests showed that the rate of pore pressure dissipation can be reliably predicted by the Terzaghi theory when the pressure-increment ratio is sufficiently large. Essentially the same value of C_v is obtained from both the compression-time curve and the pore pressure dissipation curve. For pressure-increment ratios of 0.25 or less there existed no relationship between the predicted pore pressure and those recorded from the sample. Furthermore the test results suggested that a viscous component in a thin sample results in retardation of the rate of consolidation to a greater extent than in thicker samples. This result suggests that field application of laboratory test results for secondary compression will predict rates of settlement that are too low.

Based on their survey, Leonards and Altschaeffl suggest that successful prediction of field rates of secondary compression from laboratory data hinge on the following points:

1. Successful prediction of the time when excess pore pressure is dissipated. For small load increments no useful theory is yet available.

2. Determination of a C_α value applicable to field conditions. Testing at ground temperature is most important in this regard.

3. Determination of a scaling law which adequately accounts for the drainage path.

The latter point is in definite need of clarification. Thompson and Palmer (1951) show the rate of secondary compression to be proportional to H, while Hanrahn (1954) and Lea and Brawner (1959) suggest H^2 for organic soils.

Leonards (1962) points out the effect of shearing stresses on consolidating samples. Increased shear stress produces a moderate increase in primary consolidation but the largest influence occurs in secondary volume change. This influence is easily shown by comparison of conventional one-dimensional consolidation tests with consolidation test performed in triaxial cells under an essentially no shear condition. The total volume change is greater for samples tested in the conventional consolidometer.

The author knows of no published results of long term consolidation tests in triaxial cells under anisotropic loads. Consolidation tests run under such conditions would give considerable insight into the influence of shearing stresses on secondary compression. Mohr Coulomb strength theory states that the maximum shearing stress τ in the triaxial sample can be expressed in terms of the principle stresses σ_1 and σ_3 .

$$\tau = \frac{\sigma_1 + \sigma_3}{2} \qquad 2\text{-}11$$

Therefore variation of the cell pressure σ_3 and the anisotropic load σ_1 would induce variable shearing stresses on the sample. Due to the frictional effect of the top and bottom loading caps the shearing stress would not be uniform over the full sample length. However for purposes of initial investigation into this aspect of secondary compression the error involved would not detract from the value of the findings.

2.3 Settlement Performance at Kitimat, British Columbia

The smelter of the Aluminum Company of Canada at Kitimat, British Columbia, was built on a deposit of some 350 feet of normally consolidated gravels, sands, silts and clays. The plant was built commencing in 1950 and the site preparation involved the placement of a fill having a maximum height of about 30 feet and covering an area in the first instance, about 1200 by 1600 feet. This fill has provided the major portion of the subsoil loading. Hardy and Ripley (1954) and K.L. Lee (1958) have described the site conditions and the settlement problems which developed. Complete settlement records have been kept at the site from the commencement of construction in 1950 until 1965. The maximum settlements as of 1965 are over six feet and are still continuing. The recorded settlements have been four to five times those originally estimated based on one-dimensional consolidation theory. Moreover in 1957, Lee re-evaluated the settlements using curve matching techniques, but still based on one-dimensional consolidation theory, and the settlement patterns which have developed since that time have not followed the theoretically extrapolated settlement pattern in 19 of the 21 sets of data available.

FIGURE 2-2 shows a plot for one settlement gauge at the Kitimat smelter site. The data have been taken from Hardy and Ripley (1954) and Lee (1958) and have been brought up to 1965 using data provided by Messrs. Hardy and Ripley. The significance of the settlement pattern shown in FIGURE 2-2 to the present study is that it suggests that the settlement has been predominantly secondary consolidation with the primary component being complete in a relatively short period. This assumption would account for the difficulties in accurately estimating the long term settlements in the earlier analyses. This would be inevitable if the forecasts were based on the theory of primary consolidation, while in fact the secondary consolidation was predominant. Since no pore pressure dissipation data were available at this site there was no reason to suspect for some years after construction that the settlement pattern was not in accordance with primary consolidation theory.

2.4 Settlement Performance of the Terzaghi Dam

The Terzaghi dam (previously the Mission dam) is the second major structure in British Columbia which resulted in settlements greatly exceeding the anticipated values (Terzaghi and Lacroix, 1964). The settlements of a small diversion dam which had existed at the site for some ten years prior to the construction of the larger Terzaghi dam, (completed in 1960), showed that the field C_c values were much larger than had been determined from laboratory tests. The possible influence of significant secondary settlements was not recognized when the field data from both the diversion dam and the Terzaghi dam were first analyzed.

The geology of the dam site is typical of British Columbia valleys. Steep rock walls form a narrow 600 feet wide channel infilled with glacio-fluvial fresh water sediments of gravel, sands, silts, and silty clays. The predominant compressible layer consists of 60 to 80 feet of silty clay with the natural moisture content at the top several percent above the liquid limit and further towards the bottom, well below the liquid limit. Terraces on the adjacent rock walls suggest that the site was covered with some 200 feet of overburden which has been removed by erosion.

In the report on the performance of the diversion dam Terzaghi (1957) pointed out that the field values of C_c were found to be in the order of 1.0, while laboratory consolidation tests gave C_c values in the order of 0.25 to 0.30. From piezometer data at the dam site the field coefficient of consolidation C_v was found to be about four times greater than the laboratory values. Terzaghi recognized that the compressibility of the soil was abnormally high however he could not account for the variation between the field and laboratory tests. He suggested that: "...the C_c value for this clay may decrease with increased intensity of overburden pressure..." He did not however, question the accuracy of the laboratory tests.

Based on the experience gained at the Mission dam site, Terzaghi and Lacroix (1964) were led to the following conclusions:"(a) The C_c of the clay varies erratically over a wide range, and the average is considerably greater than the laboratory average. (b) Consolidation

of the clay stratum takes place both horizontally and vertically. The rate of settlement was many times greater than the values derived by the results of consolidation tests."

2.5 Building Settlement - Long Term Records

A record of long term settlement for a group of buildings on Vancouver Island was made available to the author for discussion purposes only by Dean Hardy. When preparation of the building site began in 1901 or 1902 the formulation of the theory of consolidation was still 20 odd years away. Yet it is certain the settlements which occurred were beyond those expected by the designers. The most significant aspect of the data is the anomalous field performance compared to the theory of consolidation. The settlement record for a period of 54 years is shown in FIGURE 2-3.

The structures are found on the typical "blue clay" post-glacial deposits of British Columbia. The depths of the silt-clay stratum beneath the structures varies from 0 to 100 feet. Prior to construction of the buildings a fill to a maximum height of 20 feet was placed for site preparation.

Observation of the plots of the earliest settlement readings taken at the site suggest that only one gauge showed significant indication of primary consolidation. Most settlements since 1912 show reasonably consistent straight line plots with the log of time. Although the settlement readings before 1912 are sketchy there is considerable indication (from building damage reports) that the secondary settlements since that time have equalled or exceeded the primary settlements.

Based on these long term settlement records there does appear to be sufficient data to suggest that straight line extrapolation of plots of settlement versus log of time to estimate future settlement is valid and that secondary compression can be a continuing problem over the life of a structure. The data also indicate that secondary compression can equal or exceed the initial settlement due to primary consolidation for some silt-clays.

CHAPTER III

INVESTIGATION AND ANALYSIS OF FIELD DATA

3.1 Geology of the Site

The Duncan Lake dam is located in the Purcell trench, a one to two mile wide valley which runs from Golden to Creston, British Columbia. The elevation of the valley floor at the dam site is about 1800 feet above sea level. One half mile down the valley from the dam center-line, the Lardeau River, which flows into the Purcell trench from the north-west, joins the Duncan River. This combined flow continues about two miles where it empties into the north end of Kootenay Lake. The general surficial geology and surface features are shown in FIGURE 3-1.

Towards the end of the last glacial retreat the Purcell trench and the Lardeau valleys were filled with melt water some several hundred feet deep. As ice dams created by tributary glaciers came and went the lake level probably rose and fell over relatively short periods. Melt water from glaciers in tributary channels, such as the Lardeau, poured clays, sands, and gravels into the valley of the Duncan River. The distribution of the sediments depended on the proximity of the melt water sources and the relative lake levels. Minor ice advances and retreats during the late Pleistocene epoch caused changes in size of material deposited at any one point. Consequently

no regular sequence of deposition occurred.

The alluvial geology of the dam site is an irregular sequence of gravel, sand and silt-clay layers. The valley walls dip steeply and irregularly to a depth of nearly 400 feet. Drilling records show that the lower regions of the bedrock channel are filled with dense coarse gravel deposits. Drill logs from twelve holes located on or near the dam center line have indicated a profile of the heterogeneous alluvial deposits. The profile used as a basis for dam construction and subsequently rechecked by the author is presented as FIGURE 3-2. The drill holes penetrated on an average of approximately 250 feet into the deposits. Two deep exploration holes were sunk to 520 feet and 640 feet.

The surface deposits on the dam site are gravel and sand deposits which average about 30 feet deep. Beneath the surface layer are lacustrine deposits of thinly bedded silty sands, sandy silts and silty clays in varying depths. From observations by the author, of six Shelby samples taken from predominantly silt-clay zones, fine sand layers 1/16 to 1/2 inch thick and two to ten inches apart were found. This fine layering indicates further irregularity on a local level.

Traces of isolated terraces were found at about an elevation of 2050 feet (Dolmage, 1961). These terraces suggest that at one time up to 200 feet of overburden might have existed over the present ground surface. If some degree of over-consolidation exists then the possibility of excess settlements would be reduced considerably.

The geology report on the dam site also posed the possibility of leaching of carbonates from the silt-clay zones as an explanation

for the sensitive soil structure. Subsequent testing by the consulting geologist for the project eliminated the possibility of significant large scale leaching.

3.2 Soil Description and Characteristics

The subsurface exploration program for the Duncan Lake project was fairly extensive.The bedrock profile was established by deep bore holes and by seismic methods. The soil profile and sampling was done from deep and shallow bore holes and test pits. Shelby tube samples were usually taken at 10 feet intervals even in some of the deepest holes.

The complete drilling and testing program dhowed that the alluvial deposits consist of three main zones. A surface zone of sands and gravels covers the area to an average depth of about 30 feet. An intermediate zone of silts and silt-clays composes about 80 percent of the foundation material which would be significantly influenced by the dam structure. A bottom zone of dense sands and gravels fills the base of the bedrock channel. Detailed testing was carried out on the surface and intermediate zones.

The surface zone contains beds of gravel, sand and deposits of fairly well graded sands and gravels. The standard penetration tests showed that the deposits ranged from "loose" to "medium" dense. None were classed as "very loose". Laboratory consolidation tests on sand samples in the zone produced an average C_c value of 0.08.

The major influence of the surface zone on dam design was the high in-place permeability of the deposits. Seepage control included an

upstream clay blanket, a partial cut off through the more pervious material,downstream seepage pressure relief wells, and piezometers, the readings of which were expected to indicate the effectiveness of seepage controls.

The intermediate zone is the predominant subsurface deposit and from a design point of view it is also the most difficult zone to accommodate. The zone can be divided into two smaller sections. Silty sands and sandy silts comprise most of the zone, however two lenses about 100 to 200 feet thick of silt-clay extend from the east rock wall. These lenses gradually taper and disappear short of the west wall.

Throughout the intermediate zone thin sand strata of variable thickness and freqeuncy occur. The engineering characteristics of the intermediate zone can be summarized as follows:

Dry density - generally 85 to 95 pcf

Natural moisture content - generally 30 to 40 percent

Liquid limits - generally 23 to 41 percent

Plastic limits - generally non-plastic to 19 percent

Liquidity Index - average 1.55

Standard Penetration - generally 5 to 25 blows per foot

Unconfined compressive strength - from 1500 to 2800 psf

Effective shear strength parameters - $C=0 \quad \emptyset=33^{\circ}$ to 39°

Compressive index (from laboratory tests) - from 0.10 to 0.25

Clay content - generally from 10 to 15 percent by weight (sizes less than 0.002 mm)

Permeability - approximately 2×10^{-6} cms^2/sec at 16 to 24 tsf pressure

The most significant soil characteristic of the intermediate zone is the high liquidity index which ranges from 0.63 to 6.40 and averages 1.55. Such high liquidity indices indicate that the soil is in a "loose" state. The second important feature is the deceptively low C_c values indicated by the laboratory consolidation tests.

The bottom zone composed of dense sands and gravels extends across the bedrock floor and fills the narrowest portion to 800 or 900 feet. Due to the great depth and density of these deposits no design considerations existed.

In so far as settlement is concerned only the intermediate zone is significant. Within that zone the silt-clay layers were the most compressible. Therefore the analysis contained in this report is primarily concerned with field reaction of the soils in this zone.

3.3 Description and Construction Sequence

The design selected for the site was a zoned earth filled structure with conservatively flat side slopes and a total height of about 120 feet. The dam cross-section is shown in FIGURE 3-4. Provisions for seepage control were extensive and were described in the previous section. On completion, the dam will be about 2500 feet long at the crest and contain about 6 1/2 million cubic yards of fill. All aspects of the design were based on the requirement to accommodate large vertical settlements (in the order if 10 to 15 feet). Stage construction was decided upon because, based on the triaxial and unconfined compressive strength tests, it was estimated that primary consolidation would have to occur under stages of loading to assure stable

conditions in the foundation throughout the construction period. The rate of primary consolidation was expected to be much slower than subsequent field data showed. A three year program was expected with contract provision for an additional year if primary consolidation was slower than anticipated.

Construction of the Duncan Dam started in the spring of 1965. Winter clearing and stripping allowed the first placement of fill in May 1965. The first settlement records available started May 28 of that year. The fill placed during the first summer was predominantly on the centre third of the dam length. The procedure allowed bypass of the Duncan River near the east abutment until the tunnels in the west abutment for river diversion and subsequent release of stored water were completed. Placing of the fill in the central portion also allowed time for stripping and scaling both abutments. Loading the central section first was also expected to reduce cracking of the fill due to the large anticipated settlements in the central section. No significant change in overall fill profile occurred between November 1965 and March 1966.

Fill placement during 1966 saw the full length of the dam under loading. The diversion tunnels were completed in the early spring therefore the loading of the east third was rapidly brought up to an average crest level of 40 feet. Loading of the east third progressed until October when maximum settlements in the order of one inch per day developed. Fill placement was diverted to other sections and not continued in the east third until late November. At the end of construction in December 1966 the height of fill over the major

portion of the crest was about 110 feet.

No significant fill placement occurred again until early March 1967. Fill placement was expected to continue to completion of the design crest elevation. Settlement records used in this report reflect all loadings up to those of mid-March 1967.

3.4 Instrumentation

The particularly difficult foundation problems at the Duncan dam site demanded more instrumentation than is usual for a project of such size.

Adequate management of the project required that certain essential information be accurately known at all times. First, knowledge of magnitude and rate of movement in three dimensions was necessary. Second, pore pressures in the foundation soils had to be consistently monitored to maximize the rate of construction without endangering the foundation stability. Pore pressure measurements were also necessary to check the effectiveness of seepage controls.

The data used in this study resulted from the following instrumentation.

(a) 36 Casagrande-type piezometers set at depths from 5 to 290 feet below the original ground surface;

(b) A set of alignment hubs on the casings of the downstream seepage pressure relief wells, and on the plate type settlement gauge pipes;

(c) 2 Wilson-type slope indicators with casing down to a maximum depth of 350 feet;

(d) A P.F.R.A. type plumb-bob device to measure foundation movements

as indicated by lean in the wood stave casings of the 25 seepage pressure relief wells;

(e) 26 plate type settlement gauges set on the original ground surface and read on pipes carried up with the fill;

(f) 2 specially developed settlement sensing devices, installed in the foundation near the east end of the dam, to record subsoil movements at intervals to depths of 250 feet.

FIGURE 3-4 shows the location of the piezometers and settlement gauges on a plan view of the dam. A larger proportion of instrumentation was allocated to the eastern third of the dam to more adequately monitor the exceptionally large settlements which occurred in that area.

With the exception of the deep seated settlement gauges all the instrumentation is conventional. The unique magnetic gauges which provided most of the data for this report are described in detail in the following section.

3.5 Magnetic Settlement Gauges

The settlement readings from the original surface settlement gauges and the piezometers data for 1965 indicated that very large settlements were occurring without significant pore pressure build up. Therefore the settlement was predominantly secondary compression. This situation meant that some form of particle adjustment and or soil fabric collapse was occurring. Detailed information on the settlement patterns, especially in the more compressible silt clay layers, was masked due to the fact that the surface settlement gauges

only recorded the total settlement in the soil profile below the gauge. For adequate appraisal of the foundation stability and dam settlement the pattern of settlements was required for zones below the valley floor.

The British Columbia Research Council was assigned the task of developing a practical deep seated settlement gauge with an accuracy of $\pm$ 1 inch. Three different designs were initially produced. After further testing a magnetic type indicator was selected; a schematic view of the design is shown in FIGURE 3-5.

The magnetic settlement device consists of three basic components. The first is a sequence of magnetic marker units which are embedded at selected levels and settle with the soil. The second is a sensing device operated within an observation casing and which reacts to the magnetic markers indicating depth. The third component is the observation casing which due to its elastic properties is able to shorten with settlement without buckling or distorting.

The sensor device is a simple magnetically actuated reed switch in a capsule attached to the end of a surveyor's chain. The capsule is connected by a thin two line cable to the surface. The original switch used was 1/4 inch in diameter however some distortion of the observation casing in both S.G.-36 and S.G.-38 required that a smaller 1/8 inch diameter sensor be used. With the smaller sensor the surveyor's chain could not be used so a special cable was produced which would minimize stretching and would carry the leads from the sensor.

The magnetic marker units are rings about 2 1/2 inches in diameter and one inch thick. The markers consist of two toroidal

permanent magnets slightly separated and enclosed in plastic. The separation of the two magnets was designed to provide a sharp null point on the sensor readings. The magnetic markers move independently of the observation tube they surround. To avoid a significant difference in unit weight between the markers and the surrounding soil, layers of epoxy were added to the magnetic markers.

The observation casing used was 1/2 inch nylon tube with the elastic strain limit of about 5 percent. When the tube was prestressed to about 3 to 4 percent, settlement of 3 to 4 feet per 100 feet could occur in the tube length without buckling. Subsequent settlements in excess of the prestressing have shown that significant elastic compression has occurred without excessive buckling.

The method of installation is relatively simple. A 4 inch cased hole is drilled slightly in excess of the deepest indicator. A 600 pound weight is attached to the end of the nylon tubing and the weight lowered to the bottom of the hole. The tube is then extended a calculated amount and clamped. The casing is removed in stages as backfill and magnetic markers are placed in the hole. The tension on the tubing is maintained during pulling of the casing by alternate clamping above and below the casing length.

The magnetic gauges were designed to last until construction was complete and were expected to last for the life of the structure. The design, however, was based on the relatively low C_c values present in the fall of 1965. The increasing rates of settlement subsequent to the design and installation of the gauges has reduced the life expectancy to 7 or 8 months. The original

design did not consider the amount of elastic compression which the observation tube was capable of absorbing. The first indication that the initial prestressing had been exceeded was in February 1967 when the lower element of each gauge became blocked due to buckling. In March 1967 S.G.-38 became completely blocked by buckling in the upper regions of the tube.

Data plots comparing the upper elements of each magnetic gauge to the nearest surface settlement gauge (see FIGURE 3-6 and 3-7) indicate that accuracy in excess of ± 1 inch has been achieved. During the period of August and September 1966 both magnetic gauges were partially or totally blocked until a smaller sensor probe was developed. The extrapolated curves joining the periods when data were not available suggest an equally close agreement with the surface settlement gauges.

Another indication of the accuracy is the deviation of individual readings from the best fit curve for a given marker. The deviations shown in FIGURE 3-6 are very small; for the period prior to November 1966 the deviations do not exceed ± 1 inch and for subsequent readings, the variation from the best fit curve is negligible. The reasons for the apparent increase in accuracy of individual readings is perhaps due to operator technique and/or better surface elevation control.

The possibility that settlement of the compacted fill between the magnetic markers influenced settlement readings was eliminated by comparison of settlements to those of mechanical gauges. In the fall of 1966 two mechanical settlement gauges, S.G.-37, and S.G.-39, were installed in the foundation material. These gauges gave readings for only a few weeks before the large settlements

rendered them inoperative. S.G.-39(A) and S.G.-38 (C) were only 8 feet apart at a depth of approximately 160 feet; a close parallelism of settlement was noted over a 2 month period.

3.5 Validity of Boussinesq's Equations

Analysis of settlement results from the Duncan Lake dam was expected to require computation of some of the vertical, shearing, and principle stresses produced by successive load increments. A number of different refinements of elastic stress analysis were available. After some consideration the equations developed by Boussinesq were selected. As stresses at several different stages of construction had to be analyzed, charts published by Jurgenson (1934) could not be used.

In 1885 Boussinesq published the first adequate solution to the problem of stresses produced in an elastic medium by surface loads. Subsequently generalized solutions for the equations were published in terms of both Cartesian and spherical coordinates (Jurgenson, 1934). Boussinesq assumed a semi-infinite, homc eneous isotropic and elastic medium. The resulting equations defined all six components of stress necessary to describe the stress environment of a single element in a mass.

Subsequent work by others (Carothers 1924; Jurgenson 1934; Biot 1935; for example) resulted in some modifications of Boussinesq's original assumptions.

Pickett in 1938 published equations for determination of vertical, horizontal and shearing stresses in a homogeneous isotropic elastic solid bounded by a rigid boundary at the base. The

resulting equations were extremely complex, even for computer analysis, and were not strictly applicable to the bedrock profile at Duncan Lake.

In 1938, Westergaard published solutions based on more realistic assumptions for soil. His analysis assumed a semi-infinite material which was reinforced by many thin, closely spaced horizontal sheets which retarded lateral strain. The solutions were valid for any value of Poisson's ratio, but unlike Boussinesq's equations, the value of Poisson's ratio must be used in computation of all stresses. In comparing the Boussinesq and Westergaard expressions, Taylor (1948) stated that the magnitude of vertical stresses according to Westergaard can vary up to 2/3 of the Boussinesq value.

No definite proof exists suggesting which expression is most accurate although settlement computations based on Boussinesq's analysis are more often found to give excessive deformations (Taylor, 1948).

The soil conditions at the Duncan Lake dam no doubt more closely fit the assumptions of Westergaard than those of Boussinesq. When, however, all aspects of the problem analysis were considered at Duncan Lake, the Boussinesq equations were considered the best for the following reasons:

1. Determination of the vertical stresses did not require assigning values of Poisson's ratio.
2. The Boussinesq equation for vertical stress due to a point load is simple and therefore suitable for a computer analysis where upwards of thirty-seven thousand point loads were considered.

3. The stresses used for the original design were based on Boussinesq's analysis.
4. The stresses in the original analysis gave a range of estimated settlements which were only slightly conservative for two thirds of the dam length.
5. The trends in the settlement patterns are essentially independent of the absolute stresses and therefore do not warrant highly refined analysis.

3.6 Computerized Stress Analysis

The construction sequence described in section 3.4 dictated that settlement analysis would be necessary under four different loadings or stages. These stages were selected as follows:

Stage I	- February 1966
Stage II	- November 1966
Stage III	- January 1967
Stage IV	- Completed profile

With the exception of the final profile in stage IV the fill profile was not a constant from one abutment to the other. After some careful consideration the scaled stress profiles published by Jurgenson (1934) were eliminated as inadequate. A flexible method was needed for computing stresses at several assigned depths, for at least seventeen different gauge points, and under the four loading stages.

A computorized solution of stresses based on original Boussinesq equations met the requirements for speed, accuracy, and flexibility.

Two basic approaches to the problem of programming existed. Stresses could be computed from area loading of incremental layers. The stress due to each layer would be summed to give the influence of the complete profile. This method was investigated and rejected because the basic expression for stress at some depth due to an area load is relatively complicated and the method of expressing the limits of each elemental layer did not suggest any easy solution.

The second approach which was subsequently adopted consisted of integrating the influence of assumed point loadings, Q , created by various fill depths on the original ground surface. The chief advantages of this solution were the relatively simple Boussinesq expression for stress at a point due to a point load, Q ,

$$N_Z = \frac{3\ Q}{2\ \pi} \cdot \frac{Z^3}{(r^2+Z^2)^{5/2}}$$

where, N_Z is the vertical stress on an element at depth Z , below the point of loading, and r , is the horizontal distance from the point of loading.

The data punching was simple as depth values could be assigned to each element directly from a superimposed contour map. Finally, changes to the depth data could be made at any point with no influence on any adjacent elements or on the program.

A first approach to producing the required program was to select an element size which would cause a minimum of distortion to the true shape of the dam base, and at the same time have each of 26 settlement gauges fall on grid intersections. The size selected was a 50 x 60 foot element giving total dimensions of 38 elements in

length (2380 feet) and 24 elements in width (1200 feet) for the grid. This size adequately covered the areal extent of the dam.

A first run of the program based on the above element size showed a serious variation from published tabular and scaled values.

It was recognized that the element size was too large for use at depths above 150 feet. The error was corrected by insertion of a loop which divided each 50 by 60 foot element into 30 ten foot square elements. A check of the size assumption showed that for a ten percent or less error in the total vertical stress at a point, the distance below the loaded point had to be three times the width of the element. As no stresses less than fifty feet below ground level were necessary the ten foot square spacing was adopted. No subdivision of the depth of fill for a given large element was considered because accuracy of the fill contour map would not justify such a move. Second, the error resulting from the method is primarily a function of the element size not the depth.

FIGURE 3-8 shows the resulting computerized stress analysis compared to the scaled chart values from Jurgenson(1934). The plots are considered to compare favorably. A slight variation in results was likely due to the scaled chart assumption of an idealized profile, i.e. a consistent upstream and downstream slope.

The program was set up to compute vertical stresses at eleven elevations for S.G.-36 and S.G.-38 and at eight elevations for the remaining gauges. The total computer time for complete computation of stresses for one stage of loading was about five hours.

Relatively minor changes in the program would be necessary

for computation of any of the other five basic stresses on an element. A summation process would have to be set up to accommodate vector values.

CHAPTEF IV

LABORATORY TESTING

4.1 Introduction

To determine the influence, if any, of shearing stresses on the magnitude of secondary compression a modest laboratory testing program was set up. Samples of moderately sensitive silt-clay from Duncan Lake and sensitive varved silt-clay from the site of a samll slide on the Canadian National Railway line near Fort Francis Ontario, were selected for testing. The Fort Francis silt-clay had a natural moisture content of from 68 to 75 percent. The liquid and plastic limits were 99 and 33 percent respectively. The soil was varved and consisted of approximately equal layers of clay and silt 1/8 to 1/4 inch thick.

Conventional one-dimensional consolidation tests were performed on both soils to serve as a basis for comparison with other samples of the same soils consolidated in triaxial testing apparatus under isotropic and anisotropic loadings.

4.2 Testing Procedures

The testing program consisted of four one-dimensional consolidation tests and four triaxial isotropic and anisotropic consolidation tests. Two samples of each soil type were tested by each method. All testing was performed in a controlled temperature

and humidity room. The room temperature was maintained at $72^{\circ} \pm 1^{\circ}$ F and humidity at 49 percent $\pm$ 1 percent. The one-dimensional tests were performed according the recommended ASTM procedure. The load increment ratio $\frac{\Delta P}{P}$ used was 1, i.e. each successive load on the sample was equal to the total previous load. Simultaneous tests were performed on each soil type. Applied pressures ranged from 0 to 16 tsf. The load durations were extended to insure that an accurate secondary slope could be obtained from each load increment. At least one load increment was extended to several days to determine if there was any time dependent influence on the straight line slope of the secondary branch.

Due to sampling disturbance the results of the Duncan Lake tests were inconclusive. Therefore, results of one-dimensional consolidation tests performed for the original study of the foundation material are included. The load increment ratio of this test was not constant but varied from 1.1 to 0.125. The applied pressures ranged up to 24.05 tsf in fourteen increments. Data sheets for this test are located in Appendix C.

The triaxial consolidation tests were performed in Geonor cells equipped with standard Geonor anisotropic loading beams, (see photo 1, Appendix C). The sample size used was 1.4 inch nominal diameter and approximately 2 1/2:1 ratio of length to diameter. All samples were consolidated under a 30 psi back pressure maintained by a mercury pressure pot. Vertical change in sample height was measured with a dial gauge reading to 0.0001 inches and the sample volume change was measured in a Wykeham Farrance double 5 ml burette volume change apparatus. A photograph of the test setup is shown in Appendix C.

No means was available to indicate if lateral creep was occurring in the samples under anisotropic loads. A small wire pointer was attached to the loading cap and bent so that it was supported about 1/16 inch from mid-height of the sample. Visual inspection of the gap from outside the cell did not suggest that any significant lateral strain occurred. In view of the crude means of lateral strain control it cannot be concluded that no lateral strain occurred.

The load application sequence was the same for all samples. This procedure was intended to give some idea of the reproducibility of the test results. The samples were initially consolidated under an effective stress of 20 psi. This first load duration was extended to produce a condition of zero volume change and zero length change with time. Following the initial "zero shear stress", or isotropic consolidation, vertical pressures and cell pressures were increased to produce volume and sample height change versus log time curves for shearing stress conditions of 5 psi and 10 psi (see equation 2-11, Chapter II). The testing program was limited due to the relatively long period required for development of the secondary slope and technical problems which occurred in testing the Duncan Lake sample.

The test results of triaxial consolidation of the Duncan Lake silt-clay samples were badly distorted by leakage in the volume change burettes. The tests on these samples can be considered pilot tests at best. Discussion of the test results in the following sections

will point out the difficulties of inadequate sample volume control in anisotropic triaxial consolidation tests.

4.3 Results of Laboratory Test Program

The test results from the one-dimensional consolidation and triaxial consolidation tests are presented graphically in FIGURE 4-1 and 4-4 and in TABLES IV-1 and IV-2. In some cases best fit curves have been plotted. These curves are based on judgement resulting from other test results and reports by others (Wahls, 1962). Due to the limited number of triaxial consolidation tests the upper limit of shearing stress influence, i.e. beyond 10 psi, is only conjecture.

It should be noted that the value of C_α varies with the log of pressure in both samples of Fort Francis silt-clay and the one sample of Duncan Lake silt-clay (see FIGURES 4-1 to 4-3).

Sample test data sheets and deflection log time curves may be found in Appendix C.

TABLE IV - I (a)

FORT FRANCIS SILT-CLAY

TRIAXIAL CONSOLIDATION

K_o	Stresses (tsf)		Shear Stress (tsf)	TRIAXIAL CONSOLIDATION			
				Sample 1		Sample 2	
	σ_1'	σ_3'		Void Ratio e	C_α	Void Ratio e	C_α
1.0	1.44	1.44	0	1.550	0	1.551	0
.72	2.52	1.80	.36	1.488	.0130	1.464	.0108
.64	3.96	2.52	.72	1.445	.0238	1.420	.0309

TABLE IV - I (b)

ONE DIMENSIONAL CONSOLIDATION

Vertical Stress (tsf)	Sample 3		Sample 4	
	Void Ratio e	C_α	Void Ratio e	C_α
.33	1.974	.0104	1.891	.0065
1.00	1.781	.0105	1.711	.0107
2.00	1.514	.0156	1.545	.0160
4.00	1.330	.0157	1.355	.0181
8.00	1.147	.0170	1.156	.0151
16.00	.964	.0099	.962	.0099

TABLE IV - 2

ONE-DIMENSIONAL CONSOLIDATION

VARIATION OF C_α WITH PRESSURE FOR DUNCAN LAKE SILT-CLAY

$\Delta P/P$	Pressure (tsf)	C_α	Void Ratio e
——	0	——	.922
——	.17	.00360	.907
1.1	.38	.00374	.886
.98	.75	.00329	.867
.00	1.50	.00534	.840
1.00	3.00	.00536	.804
.50	4.50	.00505	.780
.33	6.00	.00373	.760
.25	7.51	.00373	.743
.20	9.02	.00352	.729
.25	12.02	.00383	.706
.20	15.02	.00349	.675
.165	18.01	.00392	.658
.14	21.00	.00392	.643
.125	24.05	.00272	.635

<u>Sample Calculation</u> (row 3)

$$C_\alpha = \frac{\Delta H/\text{cycle}}{H} \times (1+e) = \frac{.0017}{.859} \times (1+.886) = .00374$$

<u>Dimensions of C_α</u>

$$C_\alpha = \Delta e/1 \text{ log cycle of time}$$

$$= \frac{\Delta e \text{ (dimensionless)}}{\log t_1 \text{ (time)} - \log t_2 \text{ (time)}} = \frac{\Delta e}{\log \dfrac{t_1 \ \cancel{\text{(time)}}}{t_2 \ \cancel{\text{(time)}}}}$$

$\therefore C_\alpha$ is dimensionless

4.4 Discussion of Laboratory Test Results

The results obtained from the laboratory testing program were in some cases most useful and in other cases indicated aspects of consolidation and secondary compression which need more refined testing. The consolidation test results from tests performed by the author on the Duncan Lake silt-clays were not included because of the highly disturbed soil samples obtained and the technical difficulties with the triaxial consolidation tests.

Due to the poor quality of the samples from Duncan Lake the one-dimensional consolidation tests performed prior to the dam construction were reviewed. Of the six tests performed by Soil Mechancis Ltd. of Vancouver only one sample was a silt-clay (see FIGURE 4-3 and TABLE IV-2). A comparison of typical e-log p curves resulting from the Vancouver tests with those performed in the field indicated that sample disturbance in the latter tests was generally greater. The samples tested by Soil Mechanics Ltd. showed larger C_c values and generally larger void ratios. The void ratios of the carefully performed tests were about 1.0. This is the average value computed for the field void ratio based on equation 2-9. FIGURE 5-4 gives a comparison of the relative effect of disturbance on the e-log p curves of two samples of sandy silt.

The slope of the secondary branch for each load increment on the Duncan Lake silt-clay was found to gradually increase to a peak of $C_\alpha = 0.00536$ at 3.0 tsf vertical pressure. The resulting variation of C_α with pressure (FIGURE 4-3) is very significant. To date the only published results showing this variationare by Wahls (1962). Wahls' results were based on one-dimensional consolidation tests of a highly

pervious, calcareous organic silt naturally deposited in a post-glacial stream bed. The average specific gravity was 2.60 and the natural void ratio varied from 2.60 to 3.50. The variation of C_α with pressure may be due to the fact that the shearing stresses in the one-dimensional consolidation test do not vary directly with the applied pressures. This possibility is discussed in more detail later in this thesis.

The results of one-dimensional and triaxial consolidation tests on the sensitive Fort Francis varved silt-clay were successful within the limited objective of the test program. However, the problem of volume control in the triaxial tests continued into the second phase of the testing.

The Fort Francis silt-clay was chosen for study for three reasons. First the material had a relatively strong structure. This factor meant easier handling in preparation for testing. Second, a sensitive soil was required which would probably give comparable reactions to the Duncan Lake silt-clay. The final reason was that a block sample was obtained from a depth of about 15 feet thereby eliminating the possibility of tube sampler disturbance and major structure damage due to stress release.

The one-dimensional consolidation tests on the Fort Francis silt-clay (FIGURES 4-1 and 4-2) and the Duncan Lake silt-clay (FIGURE 4-3), all showed that the C_α values increased to a maximum value and then decreased with increased pressure or decreased void ratio. Sample 3 of the Fort Francis silt-clay (FIGURE 4-1) showed C_α increases in a "stepped" sequence. The inconsistency was likely due to sticking of

the sample load cap during the relatively slow secondary volume change. Sample 4 of the Fort Francis silt-clay showed a reasonably smooth profile of C_α variation with log of pressure. Based on the trend established by sample 4 and Wahls' tests (Wahls, 1962) the distribution curves of C_α with the log of pressure were drawn for sample 3 and for the Duncan Lake silt-clay.

Both samples of Fort Francis silt-clay (FIGURES 4-1 and 4-2) showed that the peak value of C_α occurred after the point of "preconsolidation" or the point of significant structural break-down. Considering the shape of the distribution curve of C_α with log of pressure, the evidence suggests that the majority of secondary compression occurs as structural break-down of the soil occurs. It is not possible to determine from the data available whether the structural break-down is one of simple frictional particle adjustment or the breaking of thixotropic bonds. The test results suggest that a pronounced increase in secondary compression occurs, in addition to an increase in primary consolidation, during the process of structural break-down.

In each one-dimensional consolidation test the vertical pressures were increased to 16 tsf, the load limit of the machine. At the maximum load the values of C_α were still decreasing. Wahls' tests showed that the rate of decrease of C_α with increasing stress, after the peak, is about one half that of the initial rate of increase. His tests also indicate, but do not firmly establish, that a linear rate of decrease exists. Wahls therefore suggests that the value of C_α reaches zero before the zero void ratio condition occurs. The results of both the Fort Francis and Duncan Lake silt-clay indicate a decreasing rate

of change in C_α suggesting that secondary compression exists at a decreasing rate until the zero void ratio condition exists. FIGURE 4-7 illustrates this idea.

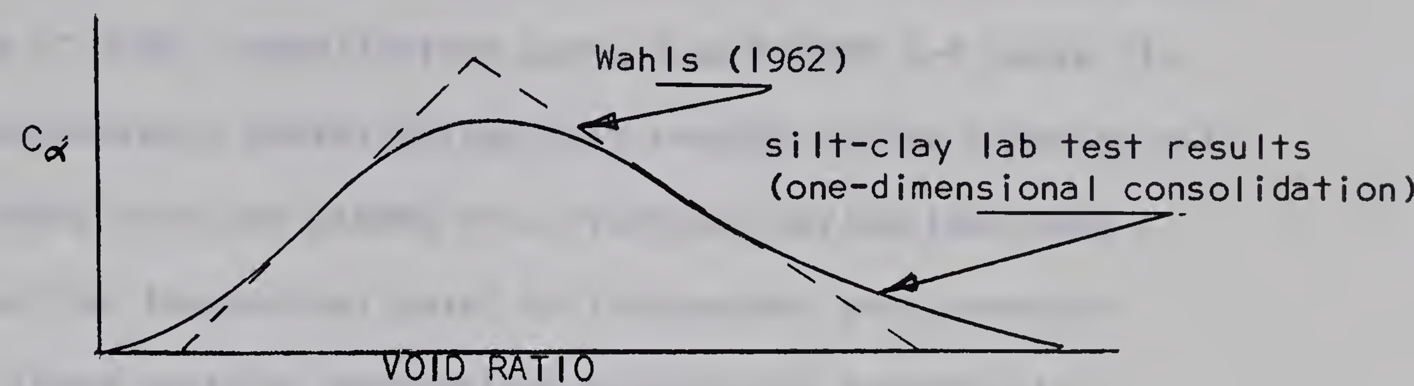

FIGURE 4-7 COMPARISON OF CURVE SHAPES C_α VERSUS VOID RATIO

Consolidation tests using loading machines with capacities in excess of 30 tsf would be necessary to confirm the behavior of C_α variation in the Fort Francis silt-clay at void ratios less than 0.80.

The object of the triaxial consolidation in the laboratory testing program was to determine if shear stress does have an influence on the rate of secondary compression. If some influence was observed then the aim was to determine some relationship between shearing stress and C_α. A complete test program was recognized to be more extensive than could be done in the eight months available, however some information regarding this aspect of secondary compression would be valuable regardless of how limited the results.

The triaxial consolidation tests (TABLE 4-1 and FIGURE 4-4) showed that essentially no secondary compression, as defined by a straight line volume change plot with the log of time, occurs when the sample is compressed by the cell pressure only. This result is common knowledge, however for research purposes it was considered necessary to

confirm the behavior for the specific soil used and also to establish a basic curve to estimate the duration of future load increments. A theoretical time curve for primary consolidation could be matched almost exactly to the $\sigma_1' = \sigma_3'$ consolidation curve (see FIGURE 4-4 curve I).

The anisotropic consolidation test results in the triaxial cell are shown in TABLE IV-I and FIGURE 4-4. Vertical deflections were measured beyond the theoretical point of 100 percent pore pressure dissipation. These vertical deflections appeared as essentially linear plots with the log of time. The data in TABLE IV-I(a) show that as the vertical stress was increased the K_o value was decreased and therefore the increase in shearing stress was proportionally greater than the increase in vertical pressure. The C_α values for both triaxial samples are almost directly proportional to the shearing stresses. The results of the triaxial consolidation tests show the same trends as those for the one-dimensional consolidation tests up to a maximum vertical stress of about 4.0 tsf. However, the results suggest that the shearing stresses in the triaxial tests for $\sigma_1' = 3.96$ tsf were greater than those found in the one-dimensional test for a similar vertical pressure.

When comparing the results of one-dimensional consolidation and triaxial anisotropic consolidation tests one should bear in mind that the shearing stress conditions in the one-dimensional test are not known. Therefore the influence on secondary compression attributed by Wahls (1962) to change in vertical pressure can equally be a direct function of shearing stress induced by the vertical loads on the sample. The results from the limited number of triaxial consolidation tests

shown in TABLE IV-I(a) suggest that this possibility may be correct.

The fundamental difference between the two test methods is that essentially no lateral shearing strain is possible in the one-dimensional test whereas there is no restriction of lateral strain in the anisotropic triaxial test. The amount of lateral strain possible in the anisotropic triaxial test is determined by the soil properties only.

The necessity of accurate knowledge of total volume change was recognized during the planning stages of the program. If lateral creep occurred then the total vertical deflection measured by the dial gauge would be disproportionate to the amount of water forced from the sample. Wykeham Farrance volume change indicators were used to measure the volume change in the sample. Due to the numerous connections and the construction of the "push-pull" stops at the base of the burettes small leaks occurred. During the entire test program the problem of leakage existed. Burette readings could be estimated to $\pm$ 0.005 ml. Due to the accuracy of the readings, leaks too small to be located showed as significant changes in the burettes. Consequently, this method of volume control was discarded for volume measurements beyond the region of 60 to 70 percent of the total volume change.

An attempt to measure small volume changes by injecting coloured droplets of kerosene into the 1/16 inch pore water tube leading from the cell to volume change apparatus proved unsuccessful because the droplets adhered to the plastic tube allowing water to by-pass. By the end of the available time for testing no effective method of volume control had been devised.

It is reasonable to expect that some of the vertical deflection recorded during secondary compression was not due to any form of densification of the soil but rather lateral strain. This condition would compare favorably to field conditions where horizontal stresses existing in the foundation usually cause some lateral creep. The creep rates resulting from the reasonably small horizontal component of the anisotropic load were not expected to be noticeable unless measured by some specially designed device. Small wire markers described in Section 4.2 showed no significant lateral strains.

4.5 Hypothesis of Increasing C_c with Pressure

Before the laboratory testing program was started settlement patterns at the Duncan dam suggested that the rate of consolidation with respect to the log of vertical stress was increasing as the fill loads increased. This means that on a plot of void ratio versus log of pressure the slope of the curve beyond the preconsolidation load (C_c value) increases. The reason for this increasing rate of settlement with respect to log of stress was not clear. Except for highly sensitive soils, such plots from laboratory consolidation tests almost invariably show the C_c value to be constant beyond the pre-consolidation pressure. Conventional settlement analyses are often made on the assumption that C_c is constant for a particular soil type. The consolidation tests of the Duncan Lake silt-clays were expected to give some insight into this aspect of the settlement problem. Careful analysis of the test results on sample 59 from Duncan Lake and test results from the Fort Francis silt-clay have suggested an explanation to the increase of C_c with load.

The theory of one-dimensional consolidation does not include the influence of secondary compression. When secondary compression does occur it results in greater changes in void ratio per load increment than is explained by the Terzaghi theory. Generally speaking these small changes are negligible and do not significantly alter the shape of the e-log p curve. If pronounced secondary compression exists in a soil its presence is usually not recognized until settlement records from the actual field loadings are plotted. In a typical laboratory test, load increment duration is terminated as soon as the t_{100} point can be determined. This step required sufficient secondary compression to obtain a straight line relationship which is extended back to intersect a straight line from the maximum slope of the primary branch of the deflection log time curve.

When stage construction is used load increments are usually placed such that some maximum pore pressure is attained, then loading is stopped until at least 50 percent dissipation of the pore pressure has occurred. If most of the settlement is due to primary consolidation this procedure works well. Settlements which have occurred in the British Columbia valleys have not been recognized as predominantly secondary compression. Therefore substantial amounts of secondary compression may be added to the relatively small amount of primary consolidation because load increments have been staged over a three year period. Because the magnitude of secondary compression is large relative to the primary consolidation, the longer the period of no load increase the larger the percent of settlement due to secondary compression. Therefore the computed values of C_c from settlement data will take into

account amounts of secondary compression not represented in the laboratory tests.

Furthermore, it has been shown in the results of the one-dimensional consolidation tests that the rate of secondary compression increases to some peak value then gradually decreases as the total pressure increases, or void ratio decreases. Sufficient data are not available from either the triaxial consolidation tests or the field records from Duncan Lake to confirm that a similar variation exists when lateral pressure can adjust to the stress-deformation conditions. However, the data available do confirm increasing C_α values with increasing pressure for both triaxial consolidation and field conditions. The present data are not comprehensive enough to show whether a peak value for C_α develops followed by decreasing values with further increases in pressure. The hypothesis will still be applicable in principle for any pattern of C_α variation with pressure. For purpose of illustration the pattern of C_α variation with pressure from the one-dimensional consolidation tests is assumed to be valid for the field conditions.

It is suggested that the non-linear relationship of C_α with the log of pressure as shown by the test results presented in this thesis can be added to the linear relationship of C_c due to primary consolidation. The resulting e-log p curve shows an increasing C_c value as found in the analysis of field data at the Duncan Lake dam.

The application of the hypothesis to a sample of Duncan Lake silt-clay is graphically presented in FIGURE 4-5. The vertical stresses of the final e-log p curve are based on the assumption that the sample

is located about 80 feet below the valley floor. The ordinates of the void ratio change due to secondary compression have been increased three times to reflect the larger C_α values calculated from the field data. The values of void change due to secondary compression are based on one log cycle which represents about 90 percent of the total increment duration. In other words, the assumption has been made that over the construction period of the dam about 90 percent of the time secondary compression has been the predominant source of settlement. This percentage is perhaps too generous in favour of secondary compression however for simplicity this break-down of primary and secondary can be easily determined from the deflection log time curves at the end of Appendix C. Primary consolidation, based on the t_{100} inflection point is roughly complete at the end of 10 minutes or two log cycles. Secondary compression for an additional log cycle of time is then added representing about 90 percent of the total increment duration.

The deflection log time plots have been separated to produce a single e-log p curve which represents only the settlement due to primary consolidation (plus that secondary compression which occurs concurrently with primary consolidation). This e-log p curve has been named the "virgin primary consolidation branch". An "in-place virgin primary consolidation curve" has been drawn parallel to the linear portion of the laboratory curve and drawn to intersect with the approximate initial void ratio in the foundation soils e_o=1.0 and at an overburden pressure of 2.3 tsf.

At the bottom of FIGURE 4-5 the void ratio change with the log of pressure is shown as an idealized incremental curve and as an

accumulation curve.

When the accumulated values of secondary compression are added to the "virgin primary consolidation curve" an e-log p curve similar to those which have developed under field conditions of loading is produced. The C_c values (about 0.56) obtained from the modified e-log p curve compare favorably with the C_c values determined at depth beneath the Duncan dam. Furthermore the curve indicates that the rate of increase of C_c will gradually become zero. The actual behavior of the curve at greater pressures is only conjecture. Similar e-log p curves based on very sensitive Leda clay would suggest that the range of significant secondary compression is limited.

The location of the peak values of C_α appears to occur after the preconsolidation load is reached. While the data on the silt-clay from Duncan Lake suggests that the above conclusion is perhaps not correct the two tests on the Fort Francis silt-clay show a definite peak value after the preconsolidation load is reached. However, as the Fort Francis silt-clay has some true cohesion the occurrence of the peak C_α value after the preconsolidation load may be due only to this fact. Therefore the distribution curve of C_α with the log of pressure should perhaps be shifted closer to the preconsolidation load of the Duncan Lake silt-clay.

Two factors account for the noticeably low void ratios obtained from the modified laboratory e-log p curve. The first and most significant is the shape of the C_α versus log p plot. The shape of the curve used in FIGURE 4-5 is based on the idealized results of the test program and Wahls' paper (Wahls, 1962). Wahls presented data

suggesting that the peak is more pronounced than the results from the test program suggest.

The second factor contributing to the low void ratio values on the modified curve is the arbitrary selection of 90 percent of the settlement time as being secondary compression. The final factor is the possible excessive 3:1 ratio of field secondary compression to the corresponding laboaratory value.

It is interesting to extend this hypothesis to the case of highly sensitive clay (see FIGURE 4-6). The resulting e-log p curve constructed in the manner outlined in the previous paragraphs is identical to the general shape obtained from consolidation tests of highly sensitive soils. Typical laboratory e-log p curves for a highly sensitive soil do not show a constant relationship between void ratio and the log of pressure as the region of the preconsolidation load is exceeded. The variation from a linear plot has been attributed to "structural break-down".

The hypothesis presented in this section suggests that the structural break-down of the soil particles manifests itself as an increasing rate of secondary compression with log of pressure up to some maximum value. Beyond the point of maximum structure break-down the effect of secondary compression decreases until essentially all the volume change is primary consolidation.

Application of the hypothesis is limited by three main factors. First, the primary and secondary branches must be separately distinguishable on the deflection log time plots. Type II curves (as shown in FIGURE 2-1) cannot be analyzed. Second, the amount of

secondary compression which occurs concurrently with primary consolidation cannot be determined as yet. A true "virgin primary consolidation branch" requires such separation of processes. Third, the rate of field secondary compression cannot as yet be adequately measured by laboratory tests.

The hypothesis does however suggest an approach to better understanding of the consolidation of sensitive soils. Based on this hypothesis it would appear that the degree of particle bonding in a sensitive soil is fundamental to the degree of abruptness in the C_α versus log pressure curve.

CHAPTER V

RESULTS OF FIELD DATA ANALYSIS

5.1 General

The results of the investigation on the settlement characteristics at the Duncan Lake Dam site are presented in this chapter. For ease of reading the results of various aspects of the investigation are listed under separate headings.

The reader is asked to bear with the fact that the results presented in this chapter are based upon several volumes of data. Presentation of this data is beyond the scope of this report.

5.2 Settlement Profiles

To appreciate the magnitude and distribution of settlements in both horizontal and vertical planes, plots were produced showing settlements along the crest of the dam (S.G. 9-18), the cross-section of largest settlement (S.G.-22, 17, and 8), and vertical settlements as recorded by S.G.-36 and S.G.-38.

FIGURE 5-1 shows the development of the settlements as recorded by the surface settlement gauges along the crest of the dam. The settlement profiles are shown after the significant pauses in fill placement which occurred during the winter and spring of 1966, and in September and October of 1966. The third profile plot records the settlements as this paper was written. Superimposed

on the profiles is the zone of anticipated settlement based on extensive consolidation test data weighted to account for the assumed disturbance of the tested samples.

The cross-section showing the largest settlements is shown in FIGURE 5-2. Due to the surcharging technique employed to dissipate initial settlements due to the water load, the most recent settlement of S.G.-22 shows a very large vertical movement. The remaining settlement profiles are created by loads placed according to the original dam design.

The vertical settlements recorded by S.G.-36 and S.G.-38 are shown in FIGURES 5-3(a) and 5-3(b). These gauges were not installed until June and August of 1966. The location of each magnetic indicator on each gauge is shown on the figures. The readings on S.G.-38 were terminated March 17, 1967 due to settlements exceeding the elastic limit of the nylon observation tube.

5.3 Determination of C_c Values

The investigation of soil consolidation characteristics considered laboratory, empirical and field data. The results of the investigations are presented in the following paragraphs.

The original settlement calculations were based on consolidation tests performed in a field laboratory located at the dam site. Subsequently a few carefully performed consolidation tests were performed in a commercial testing laboratory as a small part of an extensive strength investigation of the dam foundation. FIGURE 5-4 compares the e-log p curves for two samples of slightly sandy silt at

similar depths. One sample was tested in the field laboratory the other in the commercial laboratory.

TABLE V-I shows the range of C_c values determined from some 35 consolidation tests performed on the foundation soils.

TABLE V-I C_c VALUES BASED ON CONSOLIDATION TESTS

Soil Type	Laboratory Tests	Field Lab Tests
Sand	0.15	0.08 to 0.11
Silt-sands	0.22 to 0.24	0.18 to 0.22
Silt-clays	0.31 to 0.34	0.20 to 0.31

Only the maximum and minimum C_c values for each soil type are shown.

FIGURE 5-5 shows the relationship between the preconsolidation load (as determined by the A. Casagrande procedure) and the depth of overburden at the start of construction. Note, that with the exception of a few points near the surface, all preconsolidation loads plot below the line of overburden pressure suggesting that sample disturbance is general.

C_c values computed from settlement data of the surface gauges S.G.-15 and S.G.-16 are presented in TABLE V-2. The C_c values in column I are computed on the premise that the effective depth of settlement extends to bedrock. For the initial load increments (TABLE V-2) column I values slightly exceed those in column 2. This anomalous behavior is due to errors in vertical stress computations which tend to be more pronounced for small load increment changes. The values presented in column 2 and 2a were computed using effective depths obtained from the vertical settlement profile of S.G.-36. Both S.G.-36 and S.G.-38 showed that the effective depth was more shallow (see FIGURE 5-3) than indicated by vertical stress distribution.

TABLE V-2

C_c VALUES BASED ON SURFACE SETTLEMENT GAUGES

S.G. 16

Load Increment (Ft)	Standard Method C_c (1)	Refined Method C_c (2)	C_c* (2a)
0 - 17.5	0.09	0.06	0.06
17.5- 40	0.15	.14	.14
40 - 83	0.26	.26	.30
83 -114	0.24	.29	.34

S.G. 15

Load Increment (Ft)	Standard Method C_c (1)	Refined Method C_c (2)	C_c* (2a)
0 - 64.5	.06	.11	.11
64.5- 90.3	.20	.28	.34
90.3-116	.18	.25	.35

* Extrapolated values to give equal load duration

The results of the more refined method of computing C_c values from the field settlement gauges are shown in FIGURE 5-6. The C_c values are seen to increase with increased load. The settlement values for the last two loadings on each gauge have been extrapolated to give each load increment an approximately equal duration.

The hypothesis presented in Chapter IV suggests that the C_c value will increase to a maximum value and then decrease to some lower value as determined by one-dimensional consolidation processes.

FIGURE 5-3(c) shows a plot of C_c versus effective stress for both S.G.-15 and S.G.-16. It would appear that each curve represents a near maximum value of C_c occurring at present.

The settlement data from the magnetic gauges S.G.-36 and S.G.-38 also allowed calculation of the C_c values between each magnetic marker. Since the installation of the magnetic gauges two distinct load increments have been added. These two loading periods allowed C_c values to be computed under increasing total pressures. The results of the C_c calculations are presented in TABLE V-3 and V-4. Additional information such as total vertical stresses, void ratios, and layer thickness is also included in the tables. Graphical representation of the computed e-log p relationships are shown in FIGURES 5-7 and 5-8.

A comparison was made between the most reasonable soil profile for the immediate area of S.G.-16 and the C_c values obtained by field, laboratory and empirical methods. Tabulated values of the comparison are shown in TABLE V-5 (see page 63).

5.4 Piezometer Indication of Excess Pore Pressure

Records of piezometer readings were taken in June 1965 when only 14 of the locations selected for piezometers were used pending completion of the diversion tunnel. By June 1966 some 26 Casagrande-type piezometers had been installed. Additional piezometers were placed in the east third of the structure due to the unusually large settlement in this area.

During the first two years of construction, piezometer readings showed no significant development of excess pore pressure with

TABLE V - 3

C_c VARIATION WITH LOAD S.G. 38

Soil Stratum	Elevation Mid-point (ft)	Stratum Thickness (ft)	Aug to Nov 66 Fill-85'				Nov 66 to Feb 67 Fill 107'		
			C_c	Void Ratio	P_0*	P_1 P_0*	P_1*	C_c	Void Ratio
KJ	1778	13.70	.390	.863	8.5	11.8	13.22	.321	.851
JI	1761	19.60	.350	.874	9.0	12.2	13.35	.741	.844
IH	1747	9.68	.357	.900	9.7	12.7	13.56	.352	.882
HG	1726	30.90	.382	.898	10.9	13.8	14.83	.535	.880
GF	1704	14.16	.475	.881	11.9	14.6	15.84	.510	.863
FE	1681	31.87	.442	.893	13.2	15.8	17.13	.570	.874
ED	1657	15.80	.490	.898	14.5	17.1	18.48	.372	.886
DC	1640	17.44	.467	.896	15.4	17.9	19.31	.520	.881
CB	1626	11.84	.855	.873	16.1	18.5	19.88	.612	.856
BA	1610	20.10	.551	.900	16.9	19.3	20.73	**	**

** Settlement data not available

* units ksf

TABLE V - 4

C_c VARIATION WITH LOAD-S.G. 36

Soil Stratum	Elevation Mid-point (ft)	Stratum Thickness (ft)	July to Nov 66 Fill 80 ft				Nov 66 to Feb 67 Fill 110 ft		
			C_c	Void Ratio	**P_0	**P_1 P_0**	**P_1	C_c	Void Ratio
ML	1746	33.14	.267	.863	8.32	13.86	16.61	.266	.844
LK	1720	18.65	.268	.871	9.64	15.17	17.55	.392	.848
KJ	1704	12.62	.276	.875	10.50	15.98	18.14	.490	.850
JI	1682	17.84	.272	.885	11.72	17.11	19.20	.291	.871
IH	1670	19.67	.252	.888	12.31	17.60	19.55	.278	.877
HG	1658	5.12	.375	.869	12.95	18.14	19.94	.760	.840
GF	1649	13.40	.352	.876	13.40	18.56	20.33	.420	.862
FE	1639	6.31	.610	.829	13.98	19.06	20.78	.832	.800
ED	1628	15.69	.369	.880	14.50	19.51	21.19	.601	.859
DC	1609	22.92	.231	.910	15.68	20.54	22.09	.535	.894
CB	1576	41.56	.196	.922	17.52	22.14	23.45	.321	.915
BA	1546	17.68	.214	.921	19.14	23.55	24.95	*	*

* Settlement data not available

** units ksf

TABLE V - 5

COMPARISON OF FIELD AND LABORATORY C_c VALUES

	S.G.-36	S.G.-38	CONSOLIDATION TEST	BASED ON EMPERICAL METHOD	
				W_L%	C_c
1790′					
GRAVEL		.321			
		.741	.228	31	.245
SILT WITH SAND AND SOME CLAY	.266	.352	.240		
			.159		
			.150		
			.210		
	.392	.535	.151	32	.255
1700′	.490	.510		29	.225
			.247	32	.255
	.292	.571			
SILT TRACES OF CLAY SOME SAND	.278		.168		
				32	.285
	.760	.372		40	.350
	.420			30	.235
	.832	.520			
	.601	.612		30	
	.535	.551	.240		
1600′					
				.32	.255
SILT WITH SAND SOME CLAY	.321				
	.2[illegible]4				
				35	.293

loading. FIGURE 5-9 shows typical piezometer variations during the winter and spring shutdown in 1966. The piezometer readings are measured in feet of water above the downstream drainage ditch. The downstream level was chosen as base because it reflected the lowest static water level existing at the surface.

Pore pressure build-ups during the very rapid loading of the east portion of the dam were significant. Recorded water heads in the area of S.G.-16, S.G.-36, and S.G.-38, are shown in FIGURE 5-10. FIGURE 5-10 also shows the settlements of S.G.-16 and the fill elevation for the construction period. P-23 has been found to record the most significant pore pressure response to loading. It should be noted that after rapid loading of fill from 40 to 85 feet, the maximum excess pressure produced was 21.2 feet of water. This pore pressure corresponds to only 10 to 12 feet of fill.

A complete analysis of the adequacy of the pore pressure monitoring system at the Duncan dam is beyond the scope of this report. There are however certain important aspects which should be mentioned. The very low pore pressure readings recorded during 1965 prompted an investigation into the operation of the Casagrande-type piezometers. Tests performed by the supervising engineers at the dam site proved, to the satisfaction of those concerned, that the piezometers were functioning properly. None the less the reliability of the piezometer data must be considered in the light of piezometer response time and the suitability of the piezometer size to the length of drainage path expected in the foundation material.

Data from one-dimensional consolidation tests performed on the silt-clay zone of the dam foundation resulted in an average C_v value of 6.0×10^{-3} ft^2/min for the 16 to 24 tsf load increments. The theoretical time t for 90 percent consolidation at P-23 can be determined from the basic one-dimensional consolidation theory by:

$$t = \frac{0.85 \times H^2}{C_v} \qquad 5\text{-}1$$

where 0.85 is the time factor for 90 percent consolidation, H is the drainage path, and C_v the coefficient of consolidation. The tip of P-23 is located about 40 feet into the silt-clay zone (see FIGURE 3-2) and drainage can be assumed to be in one direction to a free surface at the silt-clay and silt-sand interface. For the condition described above, the time for 90 percent consolidation, therefore 90 percent dissipation of pore pressure, is about 155 days. The rate of dissipation of pore pressure as indicated by P-23 (see FIGURE 5-10) appears to be about 5 times the theoretical rate.

5.5 Secondary Compression

TABLE V-6 shows the tabulated results of C_α variation with depth at two different loads for S.G.-36. These values were obtained from plots of the settlement of each soil zone between the magnetic markers versus the log of time. In accordance with the definition of C_α the mid point of each loading interval was used as zero time for the plot of settlement subsequent to the load increment. As the magnetic settlement gauges were designed for an accuracy $\pm$ 0.1 feet, the lack of reproducibility between successive readings made accurate plotting of secondary settlements difficult. It also follows that

TABLE V - 6

VARIATION OF C_α WITH PRESSURE

S.G.-36

Marker	Thickness H (ft)	Period Jul to 15 Nov 66 H/cycle (ft)	C_α	Thickness H (ft)	Period Dec 66 to 20 Feb 67 ΔH/cycle (ft)	C_α
A						
	17.53	.05	.00548	*	*	*
B						
	41.21	.25	.01165	41.06	.25	.01184
C						
	22.62	.04	.00338	22.47	.18	.01510
D						
	15.39	.17	.02120	15.19	.07	.00859
E						
	6.06	.03	.00906	6.00	.02	.00602
F						
	13.08	.05	.00717	12.95	.08	.01150
G						
	5.00	.13	.0486	4.92	.01	.00375
H						
	19.27	.07	.00685	19.15	.10	.00980
I						
	17.44	.18	.01940	17.24	.13	.01410
J						
	12.30	.02	.00304	22.12	.18	.01510
K						
	18.15	.05	.00506	17.95	.07	.00721
L						
	32.19	.13	.00755	31.84	.14	.00811
M						

* No data available

as C_c values are based on a strain relationship, $\Delta H/H$, small zones such as HG and FE of S.G.-36 will reflect greater error than thicker zones. This error is especially noticeable in zone GH.

FIGURE 5-11 shows the variation of C_α with shear stress for S.G.-36. Because the influence of soil type is expected to explain some variations in the slope of the secondary compression branch, the soil profile for S.G.-36 is shown adjacent to the plots.

5.6 Summary

Chapter V has presented the results of the investigation of the field data pertinent to analysis of the settlements at the Duncan dam. Aspects considered were the slope of the settlement profiles, the C_c values obtained by field, laboratory and empirical methods, the piezometer records of excess pore pressures, and variation of secondary compression rates as shown by the magnetic gauges. The discussion of the results and their significance is presented in the following chapter.

CHAPTER VI

DISCUSSION OF FIELD RESULTS

6.1 General

Discussion of the results based on the analysis of the field and laboratory data comprising the basis of this thesis could be extremely long if not confined to the basic objectives of the test program.

The intent of the investigation was to investigate settlement behavior at the Duncan dam in the light of the Terzaghi concept of one-dimensional consolidation as applied to conventional settlement analysis. With this objective in mind the discussion will concern itself with the following aspects; comparison of laboratory and field values of the compressive index C_c, the variation of C_c with increased load, evaluation of the piezometer data, the significance of secondary compression, the applicability of the basic theory of one-dimensional consolidation to the Duncan dam foundation, and finally some possible modifications to design and construction procedures of foundations on sensitive soils.

6.2 Comparison of Field and Laboratory Consolidation Results

The results of C_c values computed from field and laboratory sources show considerable discrepancy. The original settlement computations prepared during the dam design were fortunately based on the sound conclusion

that a considerable amount of sample disturbance had occurred. Therefore, the e-log p curves of the test samples were modified in accordance with the method of extrapolating back from the zero void ratio condition (Terzaghi and Peck, 1948). To these modified curves an additional safety factor was added based on the engineering judgement of the designer.

In spite of this procedure it is indeed fortuitous that subsequent data from the deep seated gauges showed that 75 to 80 percent of the effective settlement was confined to the top 200 feet of the valley floor. The original settlement design had included the full depth of valley sediments (with the exception of the narrow canyon cut in the bedrock floor of the valley).

The laboratory C_c values of the three basic soil types in TABLE V-1 show that the largest values of C_c were from 0.20 to 0.34. The surface settlement gauges (FIGURE 5-6) give average C_c values which equal or exceed the laboratory values. This comparison indicates that within the zone of settlement, C_c values must have considerably exceeded the largest values obtained from laboratory tests. The C_c results obtained from the deep seated magnetic gauges show this assumption to be true. Calculated values of C_c reached a maximum of 0.83. In view of the fact that this maximum C_c value was based on a soil band only 6.30 feet thick, the accuracy is questionable. If however, the C_c values for the upper silt-clay zone are averaged and weighted in proportion to the soil zone height, the C_c value becomes 0.58. The value represents a 70 percent increase over the largest laboratory value and about 150 percent of the average C_c value obtained from the tested samples of silt-clay.

A similar comparison of the silt-sand zones shows that field C_c values are significantly larger than the maximum computed laboratory values.

Some of the variation in field and laboratory values can be attributed to sample disturbance. An idea of the degree of sample disturbance can be obtained from the comparison of two laboratory consolidation tests in FIGURE 5-4. Sample 59 showed significantly larger void ratios and C_c value. The consolidation test of sample 59 was performed under carefully controlled testing conditions, with an average $\frac{\Delta P}{P} = 0.4$ rather than the conventional ratio of 1.0. Test control and reduced pressure increment ratio are seen to have a marked effect on the results. The void ratios shown on sample 59 closely resemble the void ratios obtained from S.G.-36 at a similar depth, however the change in void ratio does not correspond and hence the C_c values do not correspond.

A quantitative evaluation of sample disturbance was not possible with data available from the Duncan dam. Highly refined sampling techniques were not used during the investigation of the foundation. All the soil samples were obtained by thin wall 4 inch diameter Shelby tubes. A quantitative evaluation of sample disturbance may have been possible if a few samples had been obtained using sampling equipment similar to the Swedish foil sampler.

Some 35 one-dimensional consolidation tests were performed on foundation soil samples prior to the design. None of these tests showed an abrupt decrease in void ratio with a small increase in pressure beyond the preconsolidation load. This fact can be interpreted as an indication that sample disturbance has occurred and

that the soil structure has been broken down prior to testing. FIGURE 4-3, which shows the e-log p curve for a sample of silt-clay tested at a commercial laboratory, does show a slight "break" in the generally smooth e-log p relationship. The "break" probably results more from the reduced load increment ratio (see TABLE IV-2) than from the effect of care in testing, because a reduced load increment ratio accents the relative magnitude of secondary compression to primary consolidation.

Future settlements of the surface settlement gauges at the Duncan dam will perhaps give some indication of the true shape of the in-situ consolidation characteristics. At present the slope of the e-log p curves based on S.G.-15 and S.G.-16 (see FIGURE 5-6) may represent only the initial abrupt settlement due to the break-down of the sensitive soil structure or it may represent a portion of the consolidation curve typical of sensitive to non-sensitive soils.

The results suggest that in spite of the considerable influence of sample disturbance, other factors influence the shape of the e-log p curve. The most obvious factor is the effect of pressure increment ratio. Wahls (1962) suggested that for greater agreement between field and laboratory e-log p curves, the pressure increment ratio should be as similar as possible. The average pressure increment ratio in the test sample 59 was 0.40 with a range of 1.1 to 0.125. The pressure increment values from the field based on S.G.-36 averages were 0.40 for the July to November period, and considerably less than 0.40 for the November to March 1966 period.

The effect of the reduced pressure increment is to accent the

relative influence of secondary to primary consolidation. Where soils being tested exhibit relatively large secondary settlements the pressure increment ratio should be related to the anticipated field loading ratio.

The pressure increment ratio does not account entirely for the difference between the field and laboratory C_c values. Additional factors therefore must influence the shape of the e-log p curve. These factors will be discussed in the following section. In conclusion it is interesting to note that the C_c values (see TABLE V-5) determined by the empirical relationship based on the liquid limit (equation 2-7), are for all practical purposes as realistic, if not more so, than the results of the one-dimensional consolidation tests.

6.3 Increase of C_c with Total Pressure

The rate of settlement with respect to load at the Duncan Lake dam increases with total pressure. This is shown in:

(a) FIGURE 5-6 which shows the plot of settlement versus effective stress for the surface settlement gauges S.G.-15, and S.G.-16.

(b) TABLE V-2 which shows tha values of C_c for distinct load increments at the surface settlement gauges S.G.-15, and S.G.-16.

(c) TABLE V-3 and V-4 and FIGURES 5-7 and 5-8 where C_c values between adjacent magnetic markers of S.G.-36 and S.G.-38 are presented.

The increasing C_c values can be attributed to either of two possibilities, or perhaps a combination of both. In the first instance there is the possibility that a preconsolidation load due to since eroded

sediments, once existed over the site. The geology of the area suggests that up to 200 feet of additional sediment could have existed at the site. There is however, no evidence in the data that suggests such a deposit existed. Examination of the plot of preconsolidation load versus depth (FIGURE 5-5) suggests little if any preconsolidation existed beyond the present overburden pressures. If some consideration is given to the possibility of all the samples showing lower preconsolidation pressures due to sample disturbance and stress release, a maximum preconsolidation load of 2.4 tsf (equivalent to 80 feet of sediments if γ_b = 60 pcf) appears possible. Based on the data presented in FIGURE 5-5 this load would be the maximum which could be attributed to previous deposits.

The second possibility which could account for the initially low C_c values due to low loads is soil structure. Interparticle bonding could have built up since the soil was deposited. Examination of plots of settlement and fill height versus time and the log of time for all settlement gauges on the dam center line and those at the mid-point of the upstream and downstream slopes do not suggest a discontinuity of settlement at the region of 2.4 tsf (approximately 37 feet of fill for γ_t = 130 pcf). Extremely rapid settlements do not appear to occur until some point beyond the 40 foot fill elevation.

If the amount of secondary compression is a function of the break-down of soil structure as suggested by the laboratory tests presented in Chapter IV, then the absence of prolonged secondary compression can be assumed to indicate that most of the soil structure is intact. Examination of the C_α values at the lower markers of S.G.-36

suggest that up to 4 tsf can be supported before serious structural break-down occurs. This conclusion is compatible with the e-log p curves of S.G.-15 and S.G.-16 (see FIGURE 5-6) which indicate that maximum rates of settlement with the log of pressure do not occur until the region of 4 to 5 tsf is reached. It must be remembered however, that FIGURE 5-6 represents only average stress and settlement conditions.

The effect of a proposed preconsolidation pressure can be expected to explain some of the increasing C_c values for pressures up to the preconsolidation pressure. Evidence for both the surface and deep seated gauges however, indicate that C_c values were still increasing at pressures in excess of 8 to 9 tsf. Although the rate of increase is decreasing at these larger pressures, the change cannot be easily explained in terms of an assumed preconsolidation load.

It is suggested that the increasing C_c value is a manifestation of the increasing rate of secondary compression with total pressure as suggested by Wahls (1962) and the test results in Chapter IV of this thesis. TABLE IV-6 shows the variations of C_α with increasing vertical pressure based on laboratory results of tests on a silt-clay sample of the dam foundation material. The maximum rate of secondary compression occurs at a pressure of about 3.0 tsf. Based on a comparison of one-dimensional consolidation and triaxial consolidation tests the maximum value of C_α increased if some lateral swelling was assumed. The normal consolidation process, as defined by Terzaghi, would be modified to show a gradually increasing C_c value

which would cease to increase at greater pressures. The fundamentals of this hypothesis were presented in Chapter V.

6.4 Evaluation of Piezometer Data

Monitoring of pore pressures began a few weeks after the first loading. During the construction season in 1965 the pore pressure readings showed so little response (see FIGURE 5-9) to loading that more piezometers were installed in the critical areas (S.G.-16 and S.G.-17) and deep seated magnetic gauges were developed to investigate the nature of the settlement. Subsequent loads during the 1966 season showed slightly more pore pressure response to loading but no values were at all close to the corresponding loadings.

The pore pressure responses to loading during the 1965 season ranged from essentially zero in most cases, up to a maximum of $\bar{B} = 0.1$ (P-21 on October 27, 1965). The pore pressure response is measured by:

$$\bar{B} = \frac{\Delta H}{\Delta \sigma_v} \qquad 6\text{-}1$$

where $\Delta\sigma_v$ equals the change in vertical stress at the piezometer tip, and ΔH is the excess pore pressure recorded at the tip. The pore pressure response during 1966 was more significant (see FIGURE 5-10). P-23 recorded the largest pore pressure build-up (21.2 feet of water head due to 43 feet of fill placed). Taking into account the reduction of effective stress at the piezometer tip, (located in the top silt-clay zone), the $\bar{B}$ value was roughly 25 percent. Responses from all other piezometers in the foundation of the dam were noticeably less than P-23.

The theoretical reliability of the piezometer data can be roughly appraised if the pore pressure response and the size of the piezometer tip is considered. A chart of theoretical 95 percent pore pressure response versus soil permeability is published in a report by Brooker and Lindberg (1965). Although the Casagrande-type piezometer was not tested it could be expected to respond in a manner similar to other open stand-pipe type piezometers (Geonor piezometers for example). As the shape factor (Brooker and Lindberg, 1965) for the Casagrande-type piezometer is somewhat larger than the Geonor type, response times should also be larger.

The average permeability of the foundation silt-clay based on one-dimensional consolidation tests is in the order of $1 \times 10^{-7} cm^2/sec$. The response time for a Casagrande-type piezometer for the average permeability is in the order of 2 to 4 days. In view of the possibility of sample disturbance and hence lower void ratios, the permeability in the field is likely to be greater than the value computed from consolidation tests. Furthermore the horizontal permeability (not measured in the one-dimensional consolidation tests) is perhaps 10 to 100 times greater than the vertical permeability.

In view of the fact that loadings were applied over a period of weeks and that piezometer readings were taken weekly, the effect of a 2 to 4 day delay in response is perhaps not a serious source of error. However, if pore pressure dissipation is as rapid as would appear from the rates of primary consolidation, it is possible that delay in response would not permit determination of peak pore pressure values.

A more significant source of error in the piezometer readings is the possibility of intersection of free draining sand lenses with the piezometer tip. The Casagrande-type piezometers used at the dam site are generally 24 inches long with one to two feet of sand above and below the tip. As sand lenses were found at two to ten inch intervals, the probability of one lens intersecting the piezometer is large. However, unless the sand lenses are free draining the influence on the piezometer readings will not be serious. It is not possible to determine the degree of free drainage which could occur at the piezometers.

6.5 Significance of Secondary Compression

The evidence that secondary compression has played a significant part in the amount of settlement at the dam site can be shown by both generally accepted definitions of the process. First, substantial settlements have occurred with little or no measurable pore pressures. Second, the settlement plots of the surface and deep seated magnetic markers show significant periods of roughly linear settlement versus log of time plot. The latter data is less conclusive because extended periods of no loading have not occurred since the magnetic gauges were installed.

Settlements roughly linear with the logarithm of time comprise over 50 percent of the time since construction started. These linear plots vary in slope to some extent. The long shutdown of nearly six months from January to July of 1966 allowed a period of relatively undisturbed settlement. Semi-log plots of the gauges S.G.-11 to 14 show that the rate of secondary compression began to reduce, indicating that the rate of long term settlement could occur at a smaller rate

than first anticipated. If linear extrapolations are used for predicting future settlements S.G.-16 should reflect 20 feet of settlement in the next twenty years. In Chapter II it was shown that secondary settlement in similar soil types can continue for at least 50 years.

The rates of secondary compression have been examined in the laboratory testing program. It was hoped that data from the magnetic gauges would give some indication of field behavior under stress conditions similar to those in the laboratory. Semi-log plots of the settlements of the individual soil bands defined by the magnetic markers were plotted against time in weeks. Changes in slope of the settlement plots for these bands were expected to show a significant variation with total pressure as found in the laboratory tests. Efforts were also made to develop a relationship between the theoretical shear stresses existing in the foundation with the rate of secondary compression.

FIGURE 5-11 shows the C_α values determined from S.G.-36 for two load increments. The most significant relationship appears to be with the soil type present at various depths. If C_α does vary with the total pressure then several load increments would be required to give consecutive readings from the same band. Based on two load increments the values of C_α shown in TABLE V-6 show little evidence of increase with increased pressure. According to the laboratory tests presented, the C_α values should show an increase with increased pressure. The fact that no noticeable increase in C_α is apparent indicates that there was not sufficient accuracy of marker readings or that a peak rate of secondary compression exists. Plots of the deflection log time variation due to two loads are shown in Appendix C. The

variation in successive readings is sufficient to show that the amounts of secondary compression between markers are too small for accurate measurement. The plot trends do however offer a fair indication of secondary deflections with time. Future periods of no loading will more accurately establish the secondary slopes.

If a maximum rate of secondary compression was occurring during the July 1966 to February 1967 loadings, then perhaps very little change in the C_α values would be noticed. However, as C_α is a function of pressure the variation of total pressure with depth would result in some change in C_α in a vertical plane.

Comparison of the field and laboratory values of C_α indicate that for sensitive silt-clays laboratory tests do not duplicate field values. C_α values from the one-dimensional tests (TABLE IV-1b) are about 1/3 the field values (TABLE V-6) for a similar vertical stress. The rates of secondary compression in the triaxial anisotropic consolidation tests are approximately double those of the one-dimensional consolidation tests (as shown by comparison of the Fort Francis silt-clay). This comparison suggests that use of triaxial anisotropic consolidation tests is more likely to provide the correct answer to laboratory determination of rates of secondary compression. Problems which must be considered before field C_α values are obtainable from laboratory tests are:

(a) reasonably accurate duplication of all field stresses in the triaxial sample.

(b) accurate measurement of lateral strain in the sample and this value compared to field slope indicator data.

Providing the relationship between the field and laboratory values of C_α can be determined, the method shown in FIGURE 4-5 provides a means of estimating the field e-log p curve from laboratory data.

6.6 Applicability of One-dimensional Consolidation to Settlement of Sensitive Soils

The applicability of the theory of one-dimensional consolidation to the settlements occurring at the Duncan dam must be considered in the light of the basic assumptions used in the theory's development. Furthermore, the time aspects of the theory which develop the expressions for pore pressure dissipation, hinge on reasonably accurate knowledge of drainage path length. The nature of the sediments in the foundation precludes meaningful mathematical analysis, however qualitative comments can be made.

The theory of one-dimensional consolidation assumes that all settlement can be expressed in hydrodynamic terms but at least one half the settlement at Duncan Lake can perhaps be attributed to secondary compression. Equation 2-8 shows that settlement ΔH can be expressed in terms of the vertical pressure and a soil constant C_c. The value of C_c is taken from the slope of the e-log p curve which in turn is composed of deflections due to a series of load increments. Under normal test conditions the load increment is terminated when the inflection point on the deflection log time graph can be identified. Based on recent work reviewed in Chapter II it was pointed out that current knowledge of secondary compression considers that secondary volume change occurs concurrently with, and subsequent to, primary.

The maximum rate of secondary compression is not reached until primary compression is essentially complete. Therefore, the e-log p curve produced by the consolidation test does not as a rule reflect any significant volume change due to secondary compression.

The calculations of the field C_c values were based on the total deflection due to the load increment. The piezometer data suggest very rapid dissipation of pore pressures, therefore much of the settlement was not within the limits of the consolidation theory.

6.7 Possible Modifications to Design and Construction Procedures

The results of laboratory testing and analysis of field data from the Duncan Lake dam must be reviewed with the object of deciding what procedures should be followed to more accurately provide design and construction control for heavy structures on sensitive silt-clays.

The first consideration upon initial investigation of the foundation soil is to determine whether or not it is sensitive. The settlement difficulties presented by large amounts of secondary compression appear to be particularly true of sensitive soils. A basic approach to the assessment of sensitivity is determination of the liquidity index. This approach has the advantage of being completely independent of the problems of obtaining undisturbed samples. A soil liquidity index in excess of one must be regarded as sensitive. The liquidity index of the Fort Francis silt-clay was about 0.70 yet it was unquestionably sensitive. Confirmation of sensitivity must be based on consolidation tests and unconfined compressive strength tests. All soils considered in this report have been found to be sensitive,

yet they are not alike in other aspects. The effect of varving, deposition in fresh water or salt water, clay content, and the effect of preconsolidation will introduce additional factors into the consolidation characteristics. The effect of sensitivity therefore, must not eclipse other equally important factors.

Once is has been determined that the soil is sensitive, the problem of adequate assessment of the field consolidation characteristics is faced. There appear to be two essential steps to this aspect of the investigation. First, the relative proportion of primary and secondary consolidation must be determined and second, the rate of secondary compression must be established. The one-dimensional consolidation test does not appear to be satisfactory for the job. Results based on the one-dimensional consolidation test have failed to predict settlements at Kitimat, the Mission dam or Duncan Lake. The one-dimensional consolidation test can perhaps give some idea of the relationship of C_α with pressure. This relationship suggests that it can be very important to the ultimate settlement analysis (see FIGURE 4-4). The laboratory determination of the magnitude of C_α is not adequate at present, however the use of triaxial anisotropic consolidation tests appears to offer the most primise.

The yet unsolved problem in use of the triaxial cell for consolidation testing is determination of the necessary stress environment to duplicate field rates of secondary compression. Results from the triaxial tests suggest that in addition to shear stresses a certain amount of lateral strain and/or collapse of structure must be present to account for additional secondary settlement.

The rate at which consolidation of soils subject to large secondary settlements may proceed, presents the problem of stage construction in projects such as the Duncan Lake dam. If the decision is made on the basis of pore pressure build-up, clearly then stage construction is not necessary. Maximum pore pressure reactions of 0.25 do not as a rule constitute a serious strength reduction. However when settlement rates reached 0.1 feet per day in the fall of 1966, the possibility of liquification of the foundation presented a serious problem. Rates of construction under such conditions must be controlled on the basis of reliable data on the rate of strain necessary to induce liquification. Again the triaxial anisotropic consolidation test appears to offer the most reasonable approach to determination of a liquification criterion.

Provided rates of strain do not reach the point where liquification occurs, the settlements can be reasonably expected to follow e-log p relationships as suggested in the hypothesis in Chapter IV. Plots of C_c versus average total stress for S.G.-15 and S.G.-16 (see FIGURE 5-3(c)), suggest that the near maximum C_c values are presently occurring. Future loading due to the weight of reservoir water hopefully will produce somewhat lower C_c values.

A final problem which seriously influences the settlement analysis of structures similar to the Duncan Lake dam is the depth of effective settlement. The depth of effective settlement based on Boussinesq's or Westergaard's expressions is at least twice that indicated by the deep seated settlement gauges (see FIGURE 5-3). A reappraisal of the applicability of existing elastic stress concepts to deposits "funneled" by a sloping, rigid wall may be necessary.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary of Objectives

The object of this thesis was to investigate and explain the anomalous settlement behavior of the foundation soils at the Duncan Lake dam in the light of conventional consolidation theories. The investigation was supported by a modest laboratory testing program of one-dimensional consolidation and triaxial anisotropic consolidation tests.

7.2 Conclusions

1. The anomalous settlement performance of structures situated on deep, sensitive, loose alluvial deposits of silts and silt-clays in the fjords of British Columbia, including the Duncan Lake dam, the Terzaghi dam, and the Kitimat Smelter site, is due to the fact that the C_c value of the soil increases with the increase in total load.

2. The increase in C_c values with increasing load is likely due to secondary compression becoming relatively large compared to primary consolidation. The fact that laboratory C_c values do not correspond to field C_c values can be attributed to the influence of sample disturbance and the fact that conventional one-dimensional consolidation tests do not reflect the field magnitudes of secondary compression.

3. Present conventional methods of laboratory testing for consolidation characteristics of the soils described in Conclusion I do not permit accurate determination of secondary compression characteristics. The C_α values determined from the field were as much as 3 times greater than C_α values based on standard one-dimensional consolidation tests, but likely only slightly larger than C_α values obtained from the pilot triaxial anisotropic consolidation tests.

4. Conventional settlement analysis based on one-dimensional consolidation using C_c values from standard laboratory consolidation tests cannot be expected to give accurate estimates of the settlements in the field if secondary compression is relatively large compared to primary consolidation. Conventional settlement analyses are based on C_c values, assumed to apply at the end of primary consolidation, and therefore are independent of time. However, the component of settlement due to secondary compression is time dependent. A settlement analysis based on C_c values assumed to be independent of time therefore cannot be expected to give rational predictions.

5. To adequately estimate settlements, including long term settlement for structures on deposits in fjords in British Columbia such as the Duncan Lake dam, the Terzaghi dam and the Kitimat Smelter site, it will be necessary to distinguish between the primary and secondary components of settlement and analyze each separately. The present difficulty is to obtain a value of C_α which truly represents field behavior.

6. A quantitative evaluation of the effect of sample disturbance on the shape of the e-log p curve and on the laboratory rates of secondary compression could not be made from the existing data on the Duncan dam.

7. The laboratory tests confirm the results of Wahls' work that the C_α values from laboratory consolidation tests are not constant and in one-dimensional consolidation tests they increase to a maximum value and then decrease as the total pressure increases. The field C_α values from Duncan Lake showed some increase with increasing load.

8. A hypothesis which allows the summation of C_α values with the e-log p curve due to primary consolidation presents a rational explanation of the consolidation process in sensitive silt-clays as total pressure exceeds the bond strength of the soil structure. The hypothesis allows independent manipulation of the e-log p curves resulting from the primary consolidation and secondary compression of the soil structure.

9. Stage construction of structures built on the soil described in Conclusion I, cannot be controlled on the basis of excess pore pressure build-up but rather on the basis of the maximum rate of strain necessary to induce liquification of the sensitive soil. At present there is no laboratory procedure for determination of this maximum rate of strain.

10. Deep seated magnetic settlement gauges indicate that 75 to 80 percent of the total foundation settlement at the Duncan Lake dam is confined to the top 200 feet of deposits. In view of the consolidation characteristics of the foundation soils, this situation appears contrary to both Boussinesq's and Westergaard's theories of elastic stress distribution. The reason for the occurrence may be either the result of "funneling" the deposits as the valley cross-section becomes smaller with depth or, degrees of non-homogeneity and anisotropic characteristics which greatly exceed the assumptions of the theories.

11. Digital computer determination of stresses at depth under asymmetrical surface loadings result in substantial savings in time and give more accurate results as compared to results determined from published stress coefficients.

7.3 Recommendations for Future Research

Based on the results of this thesis four basic problems must be analyzed during the course of future research and testing in the field of settlement analysis of sensitive soils.

1. Studies should be undertaken to determine the nature of secondary compression. Is it primarily a function of shearing strain or a combination of magnitude of principle stresses and shearing strain?
2. Laboratory tests should be undertaken to develop test procedures that will permit determination of C_α values as they would occur under field conditions. Triaxial consolidation tests under anisotropic loading conditions appear to present the most encouraging approach at present.
3. Further evaluation of the relationship of sample disturbance to the anomalous settlement behavior of the Duncan dam should be made. The most highly refined sampling techniques available should be used. Both conventional one-dimensional consolidation tests and triaxial anisotropic consolidation tests should be performed on the undisturbed samples.
4. The anomalous vertical distribution of settlement as recorded by the deep seated magnetic gauges should be investigated in the light of existing theories of elastic stress distribution and under more refined methods such as a finite element analysis incorporating the entire foundation zone and the rigid, funneled, valley walls.

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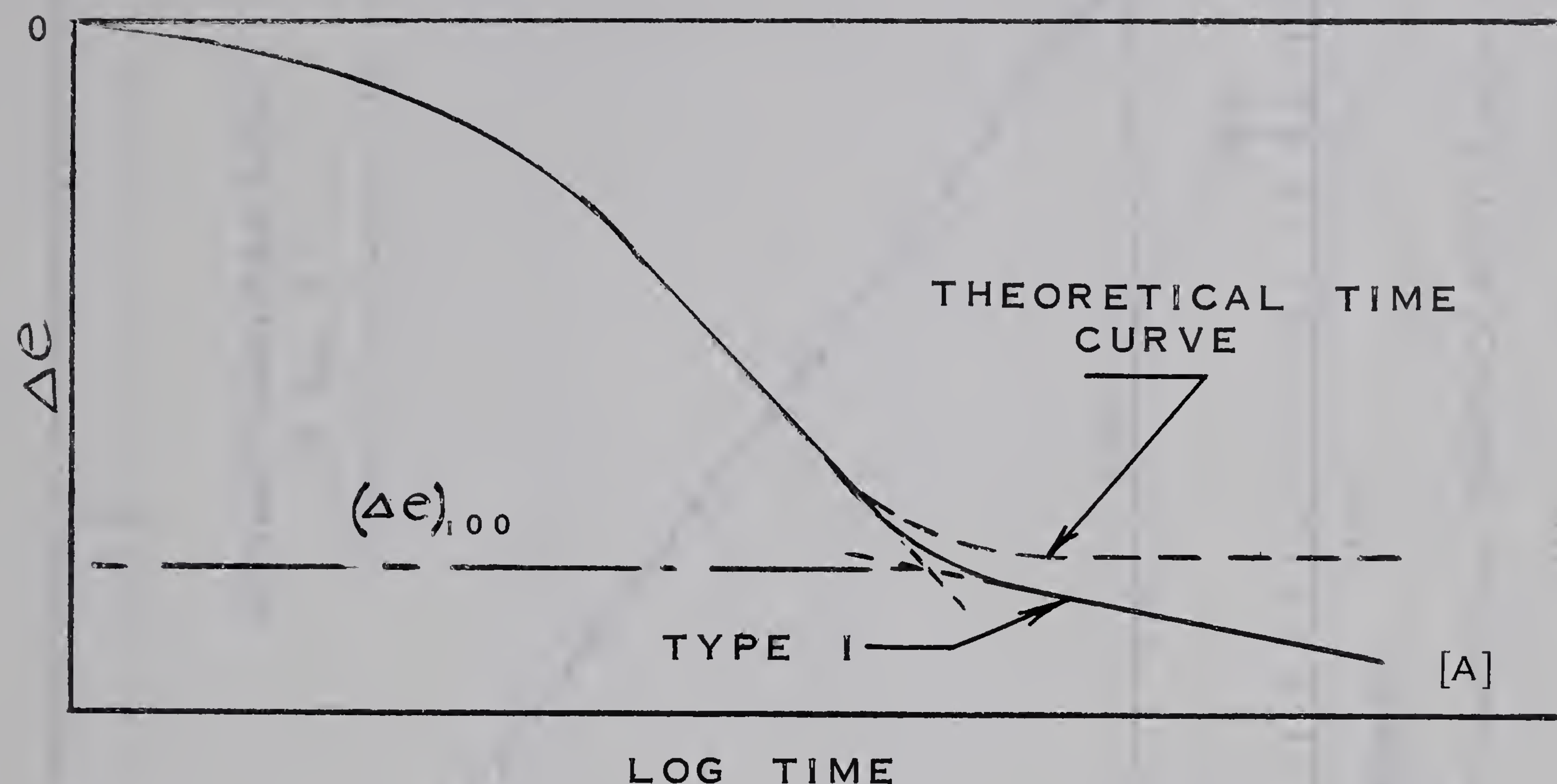
Appendix A

FIGURES REFERRED TO IN BODY OF THESIS



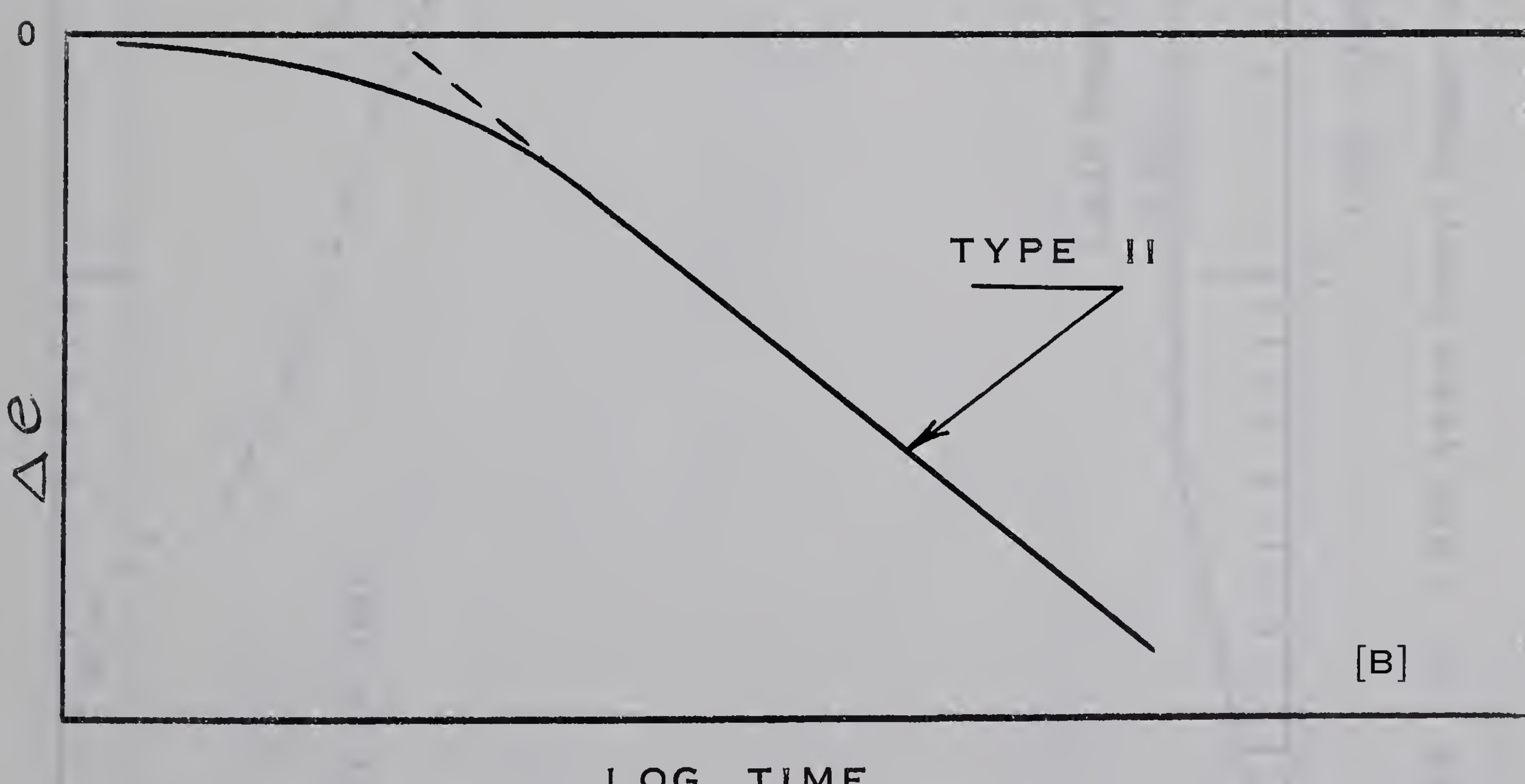


FIGURE 2-1 [AFTER WAHLS, 1962]
TYPICAL VOID-RATIO CHANGE VERSUS
LOGARITHM OF TIME CURVES

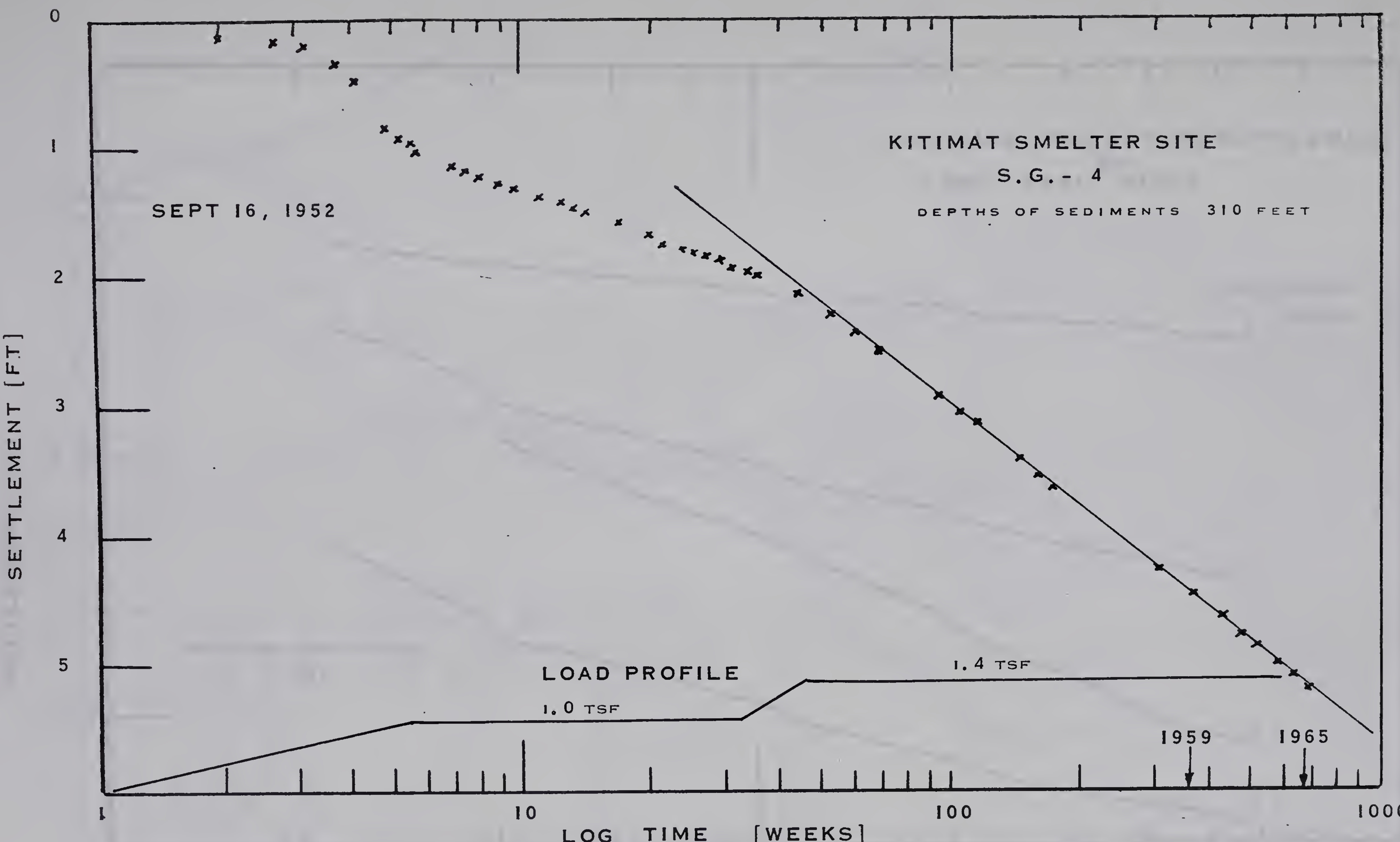


FIGURE 2-2. LONG TERM SETTLEMENT KITIMAT SMELTER [SURFACE SETTLEMENT GAUGE]

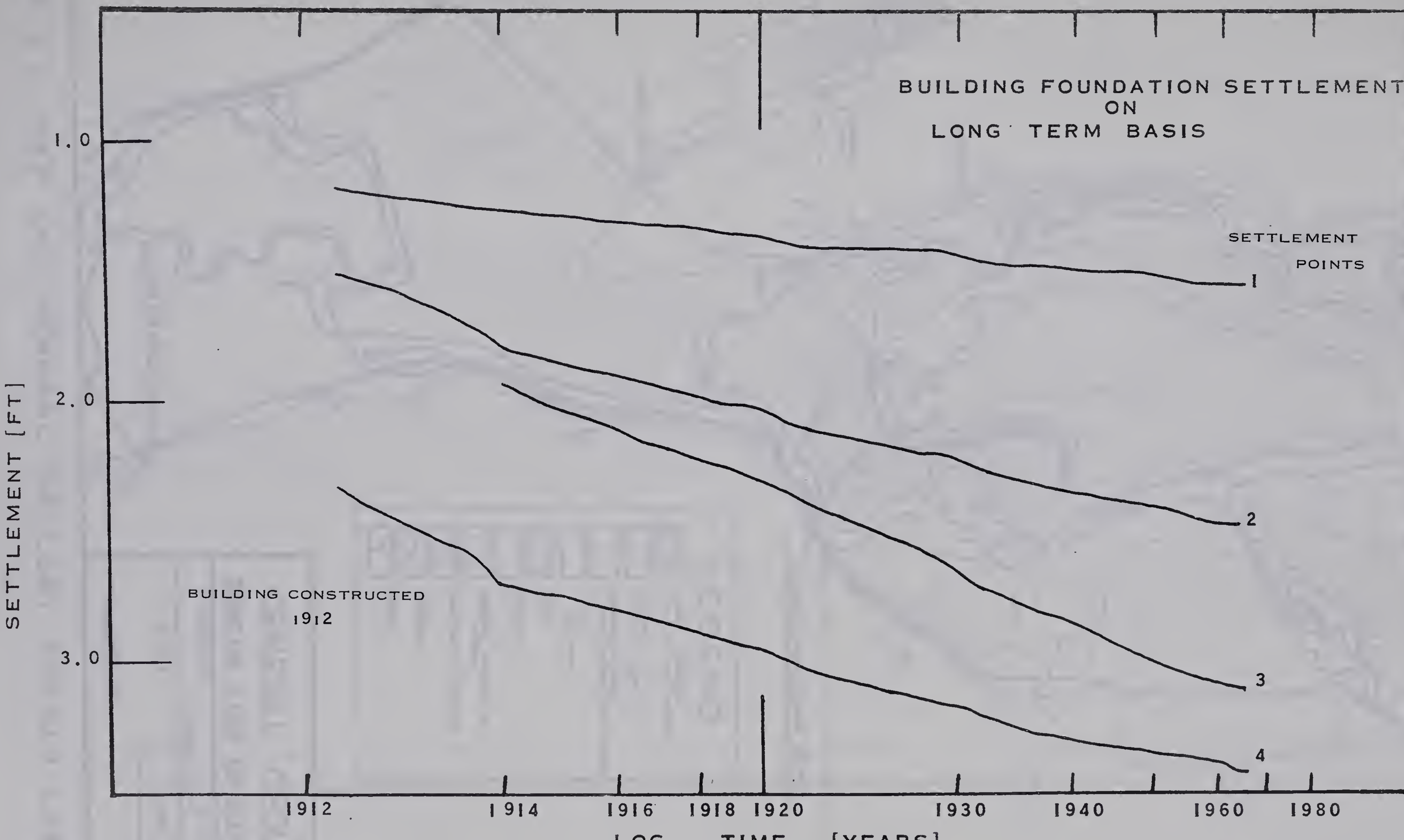
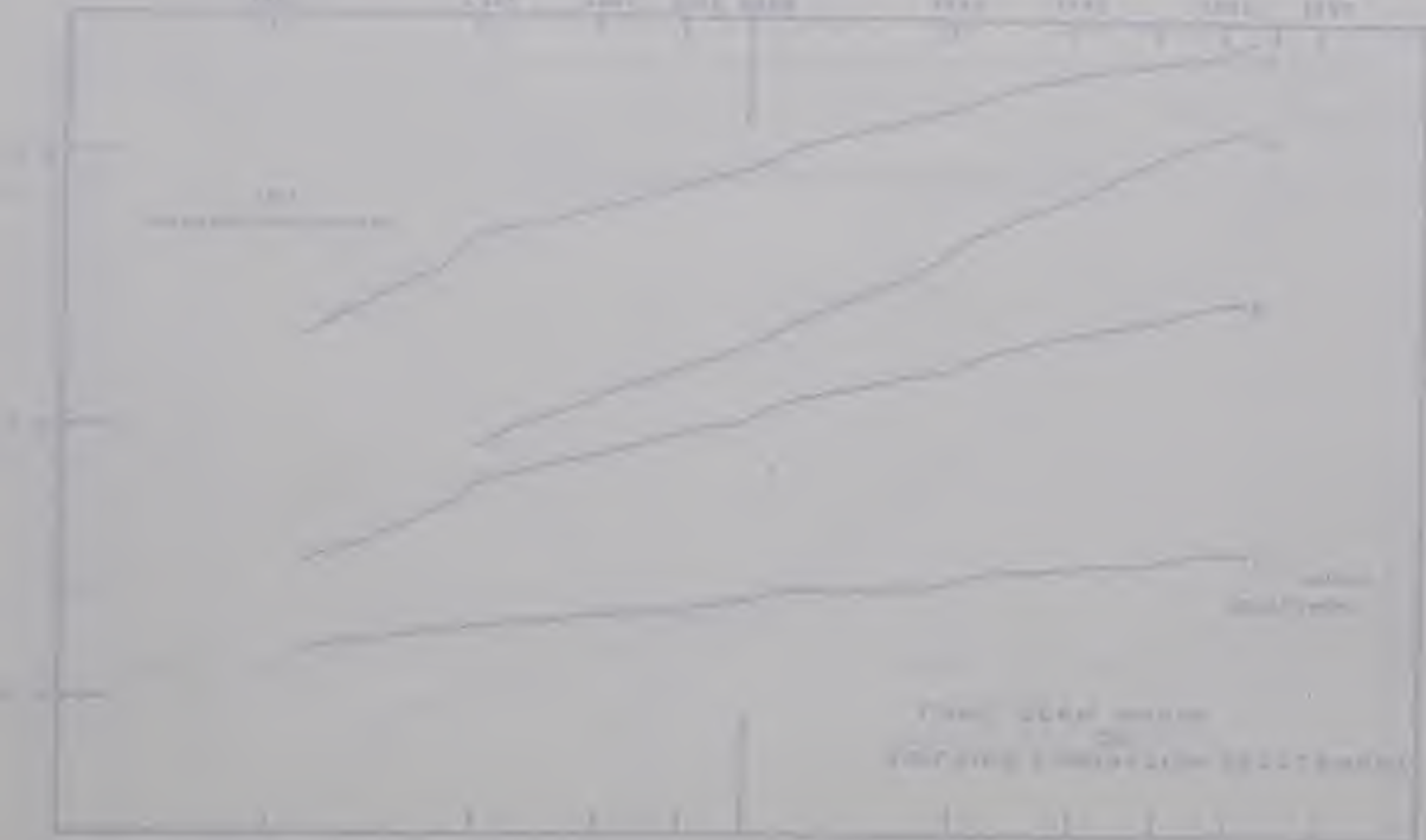


FIGURE 2-3 LONG TERM SETTLEMENT VANCOUVER ISLAND, B.C. [SURFACE GAUGES]



A-1

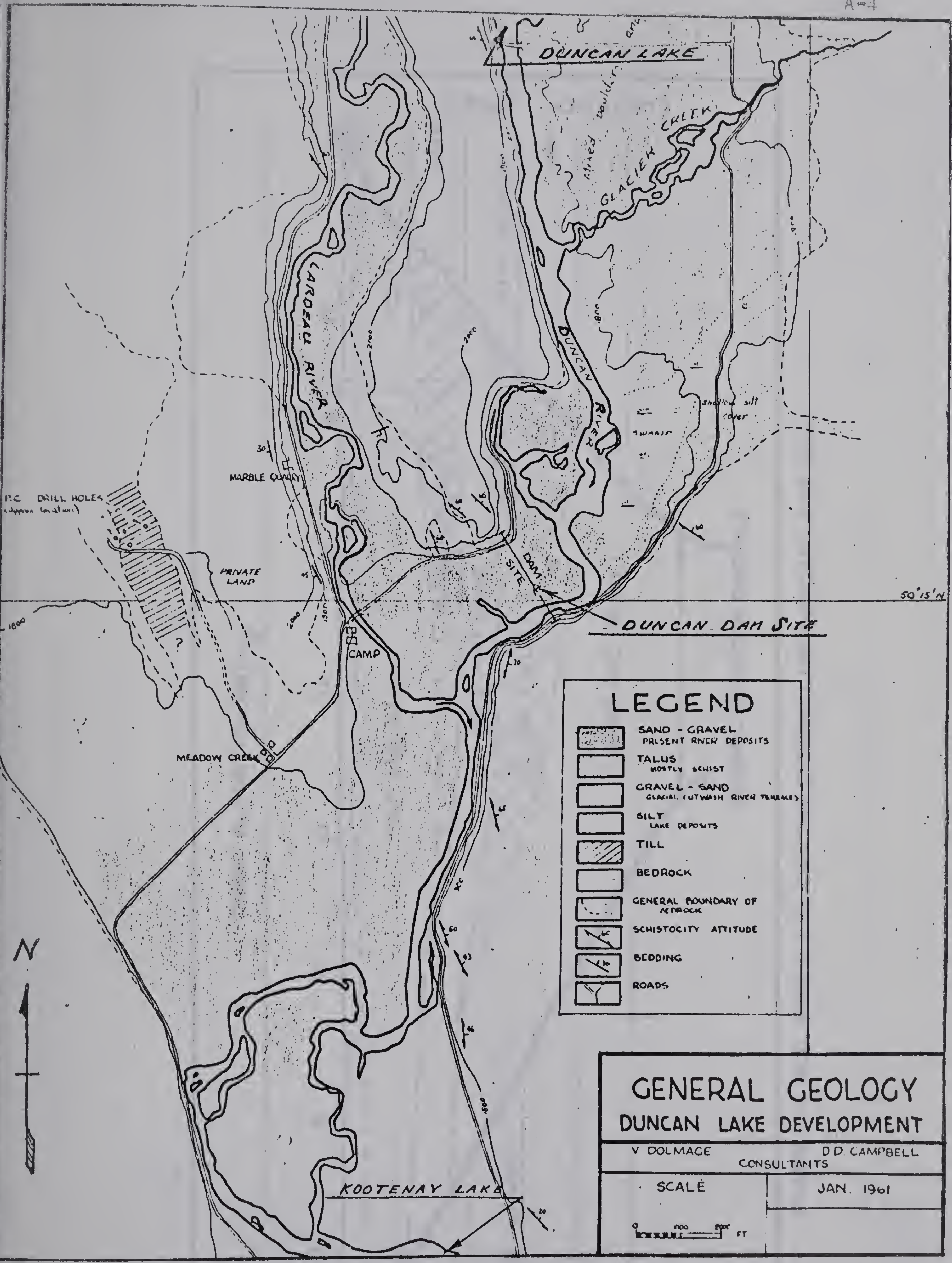


FIGURE 3.1 MAP AND GENERAL GEOLOGY DUNCAN LAKE DAM

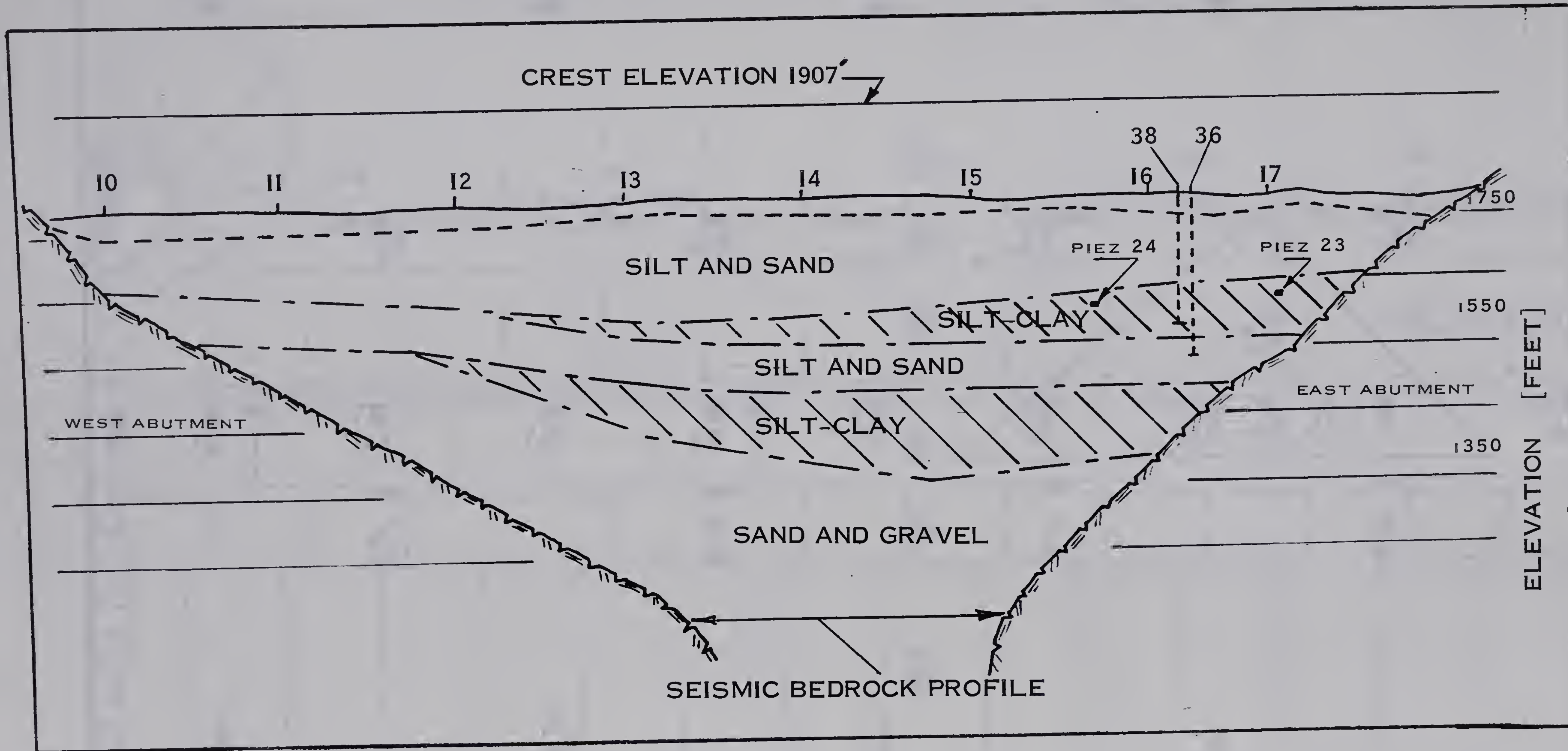
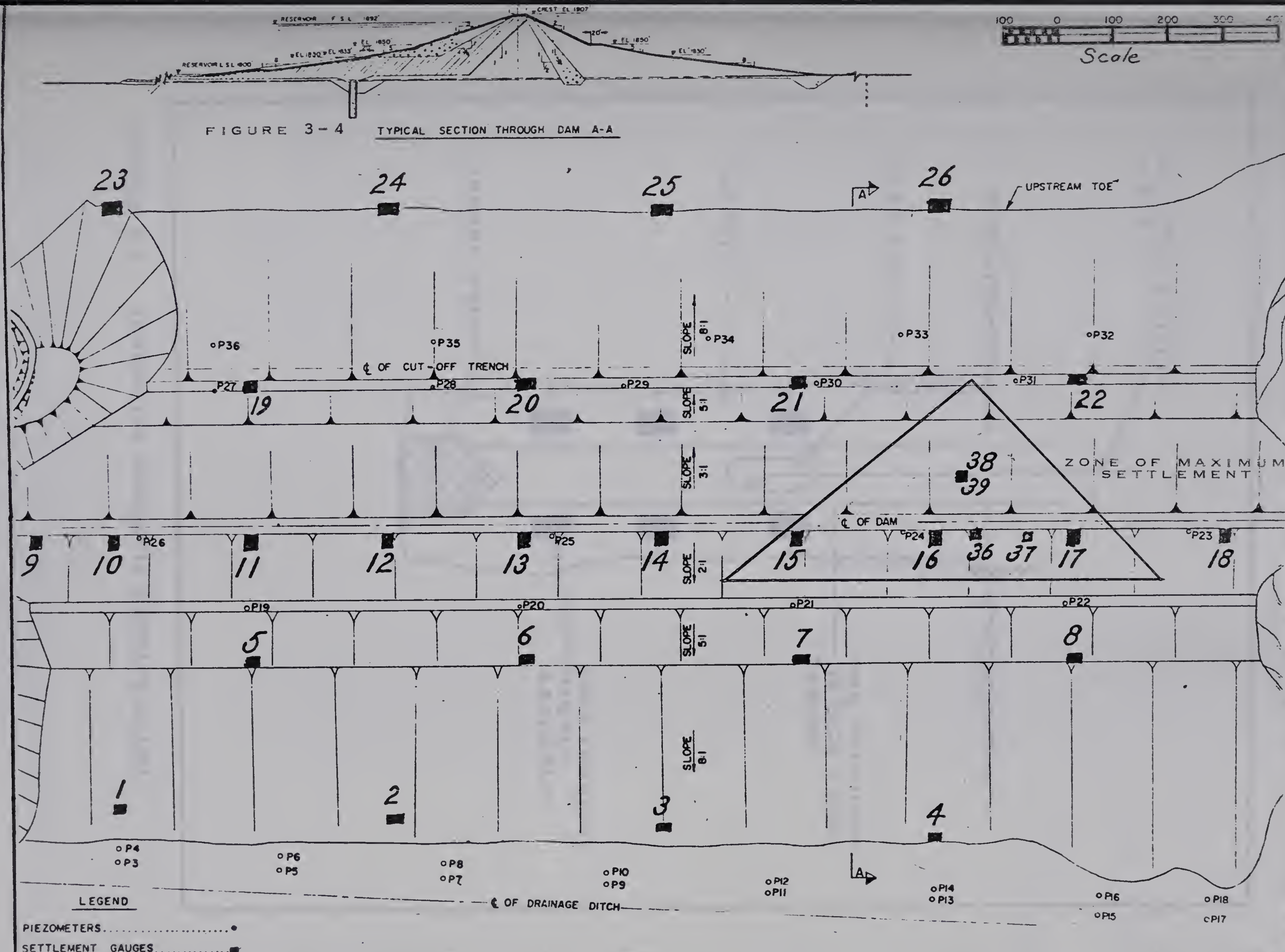


FIGURE 3-2 SOIL PROFILE ALONG THE CENTER LINE DUNCAN LAKE DAM



FIGURES 3-3 AND 3-4 PLAN VIEW OF SETTLEMENT GAUGES AND PIEZOMETERS WITH CONSTRUCTION PROFILE AS INSERT DUNCAN LAKE DAM

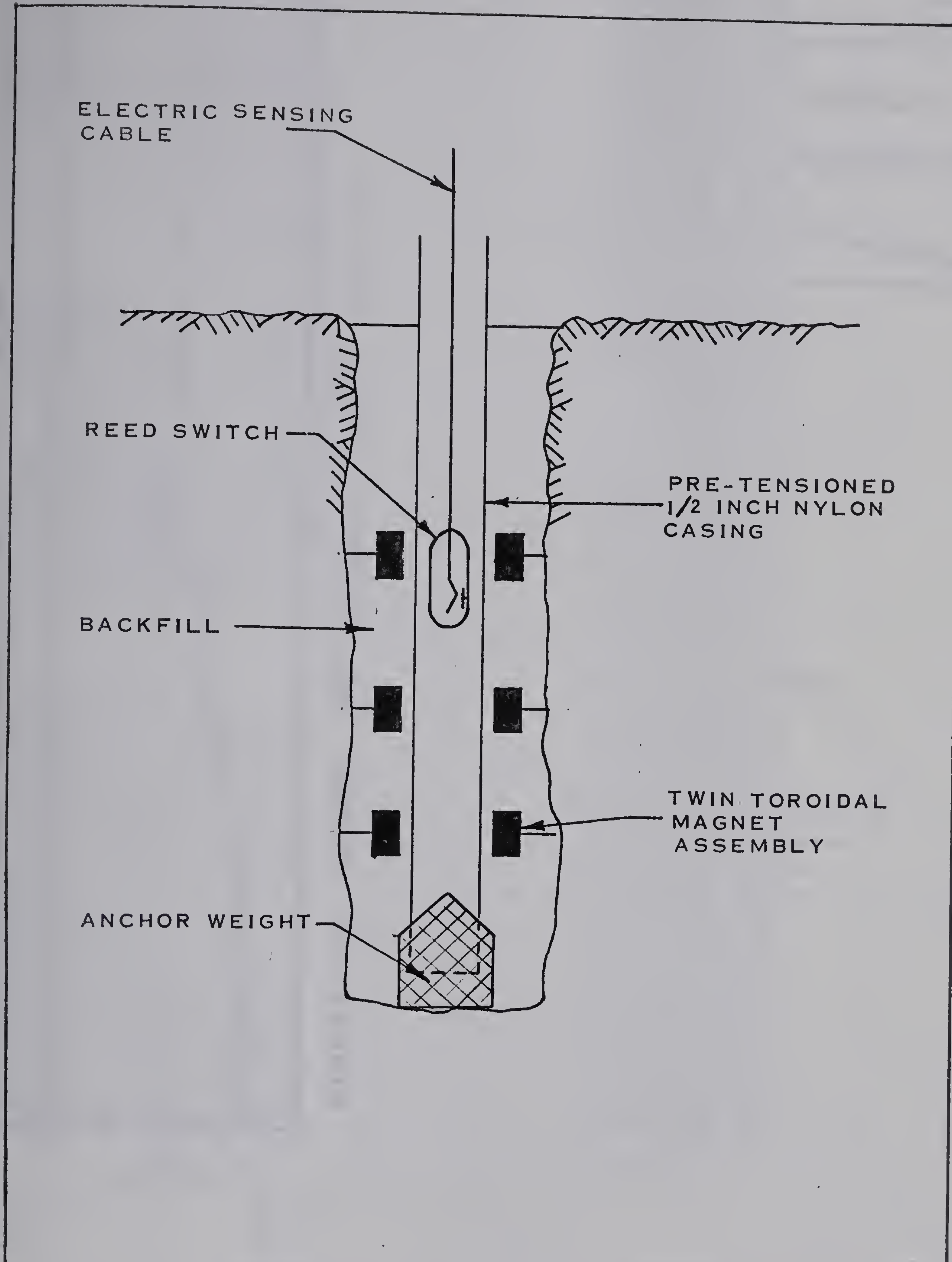


FIGURE 3-5 SCHEMATIC MAGNETIC SENSING GAUGE

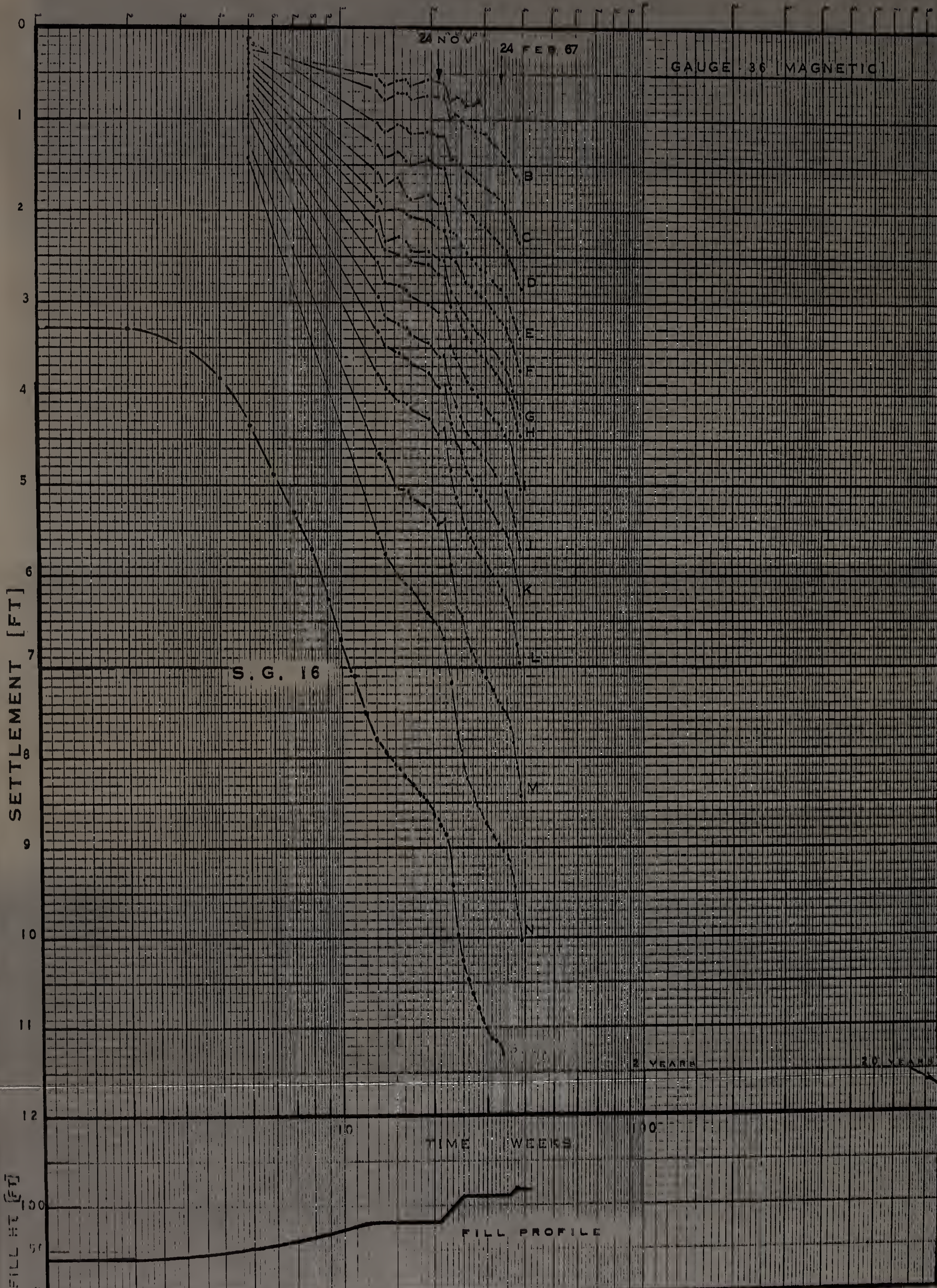


FIGURE 3-6 MARKER SETTLEMENT PLOTS S.G. 36

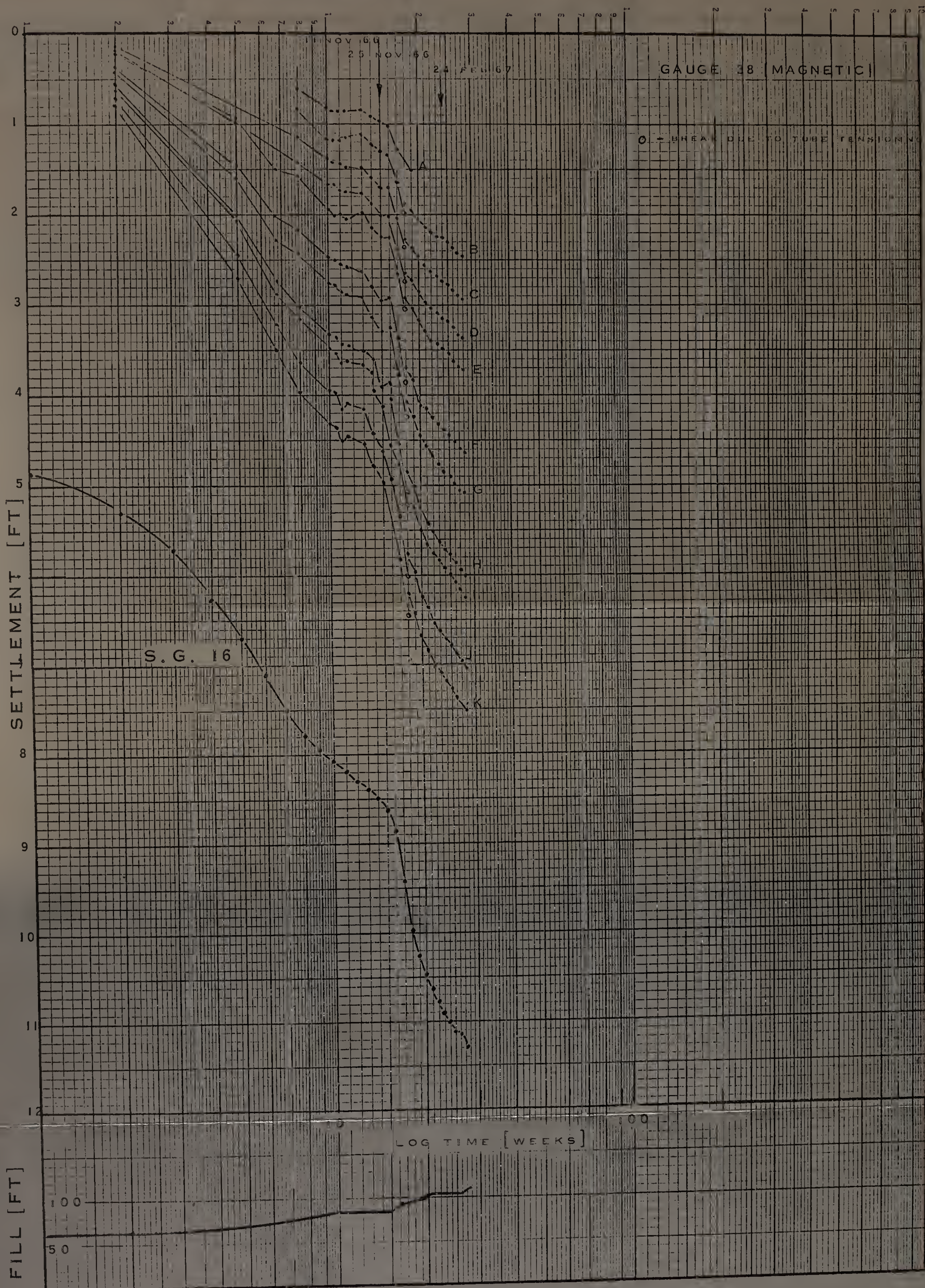


FIGURE 3-7 MARKER SETTLEMENT PLOTS S G 38

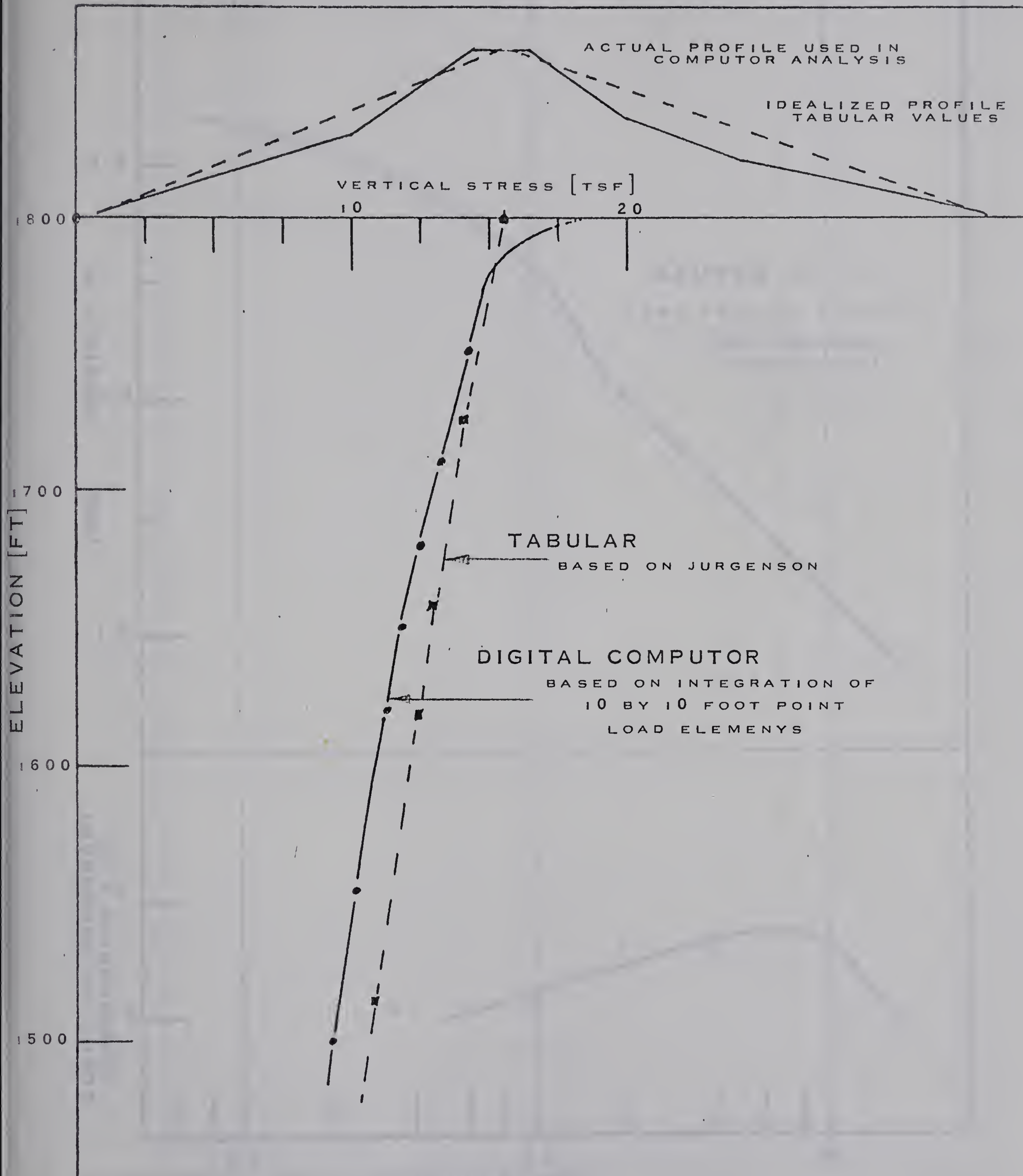
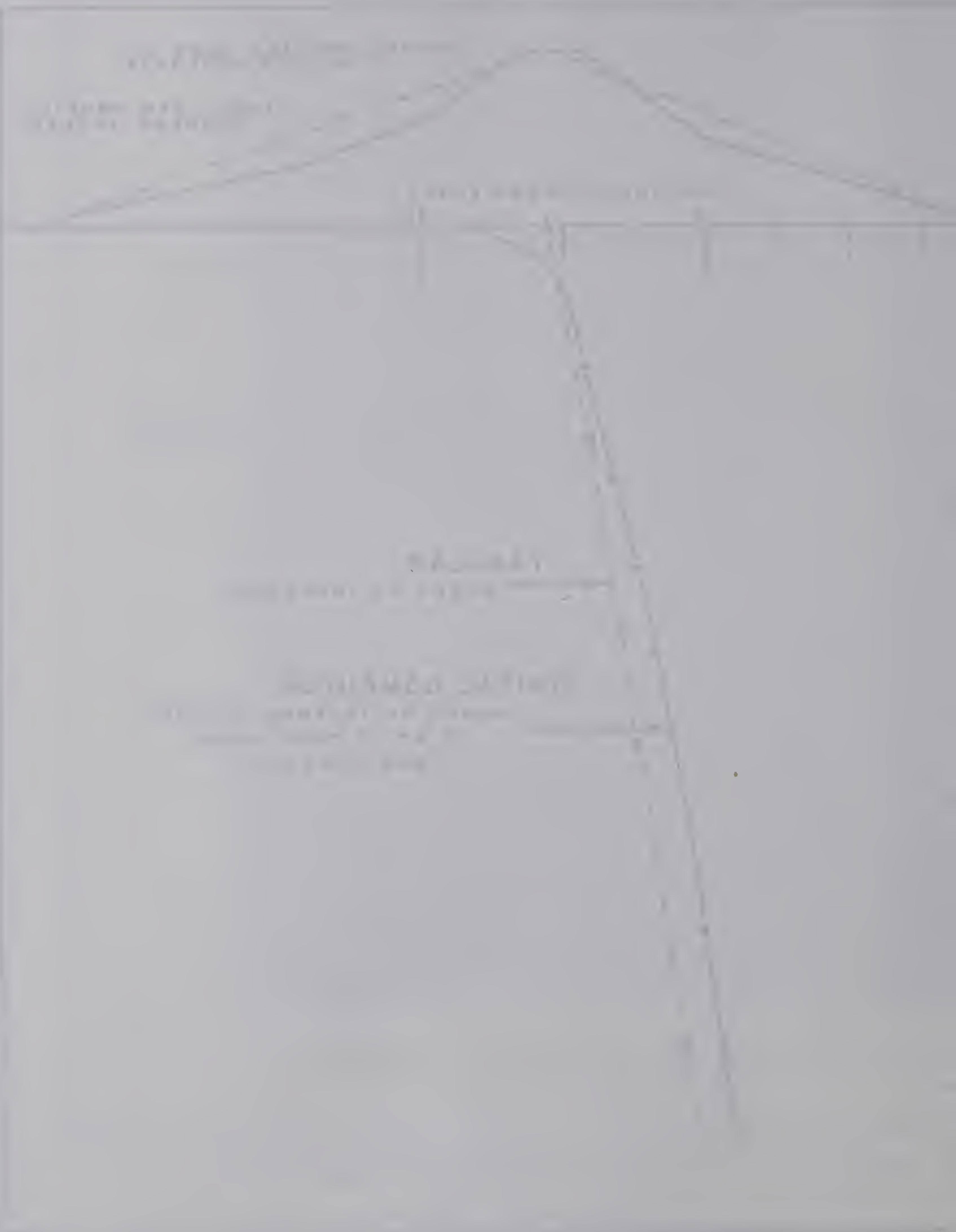


FIGURE 3-8 COMPARISON OF TABULAR AND COMPUTORIZED VERTICAL STRESSES



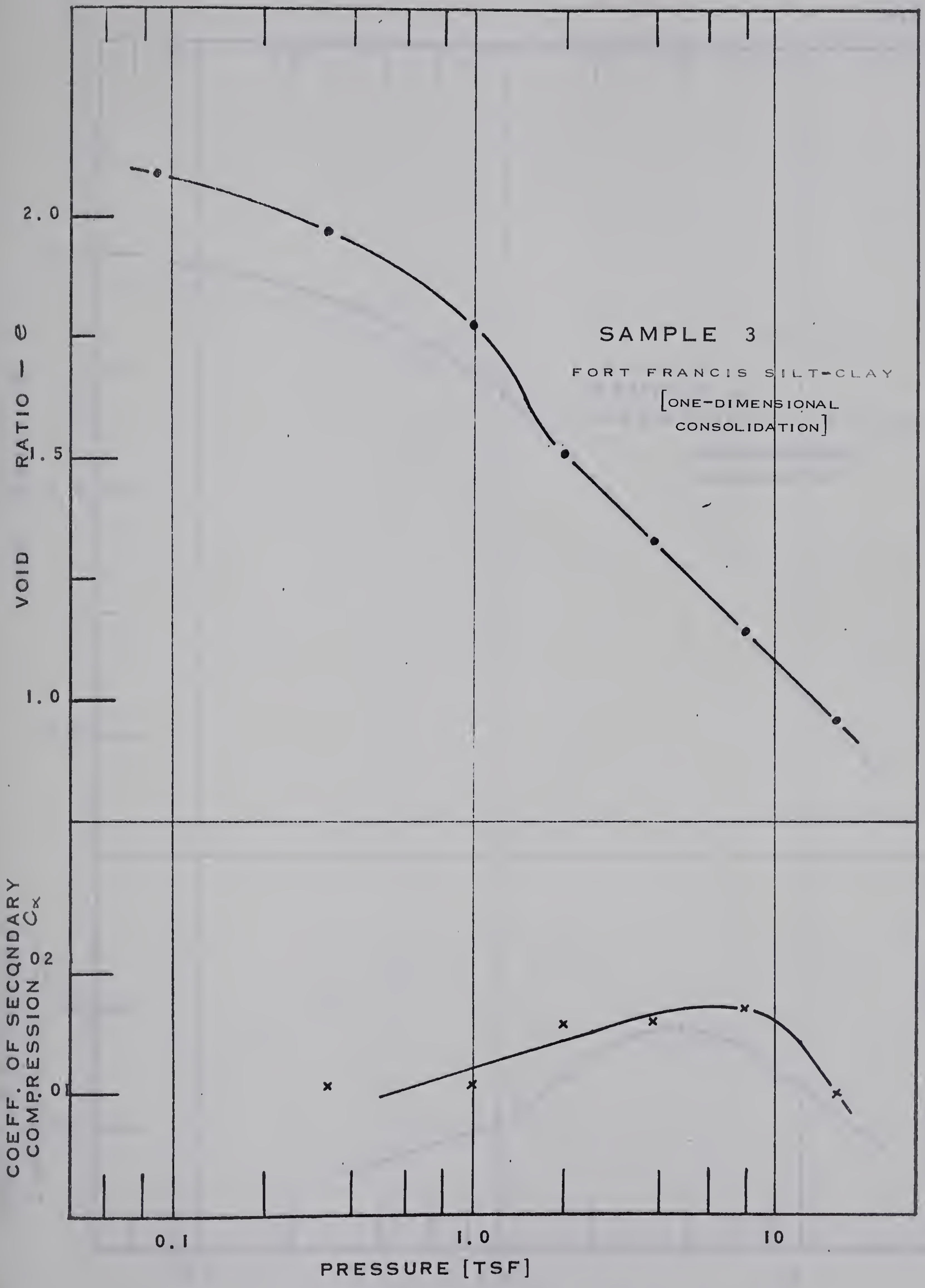


FIGURE 4-1 SAMPLE 3 FORT FRANCIS SILT-CLAY CONSOLIDATION TEST RESULTS

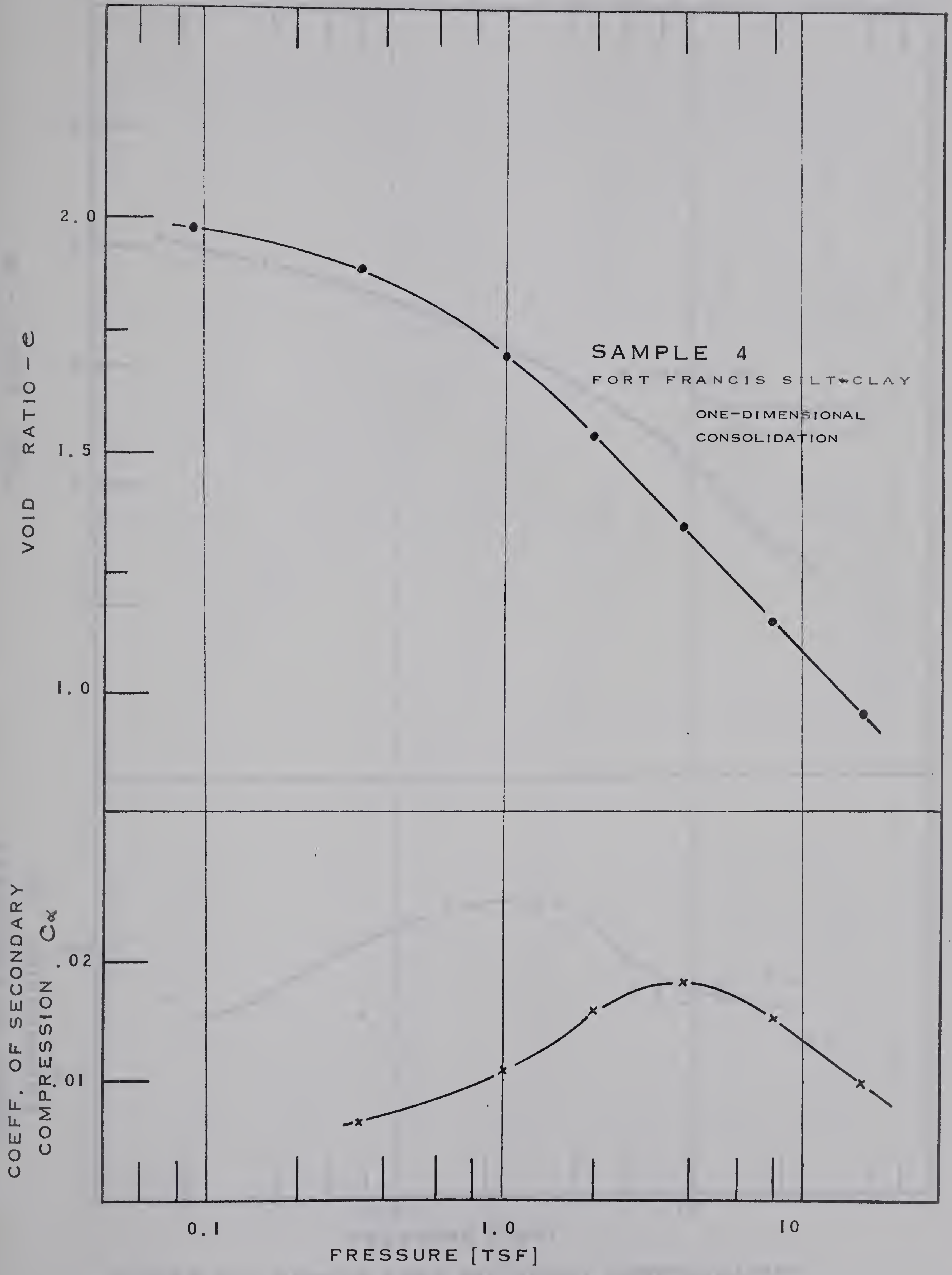
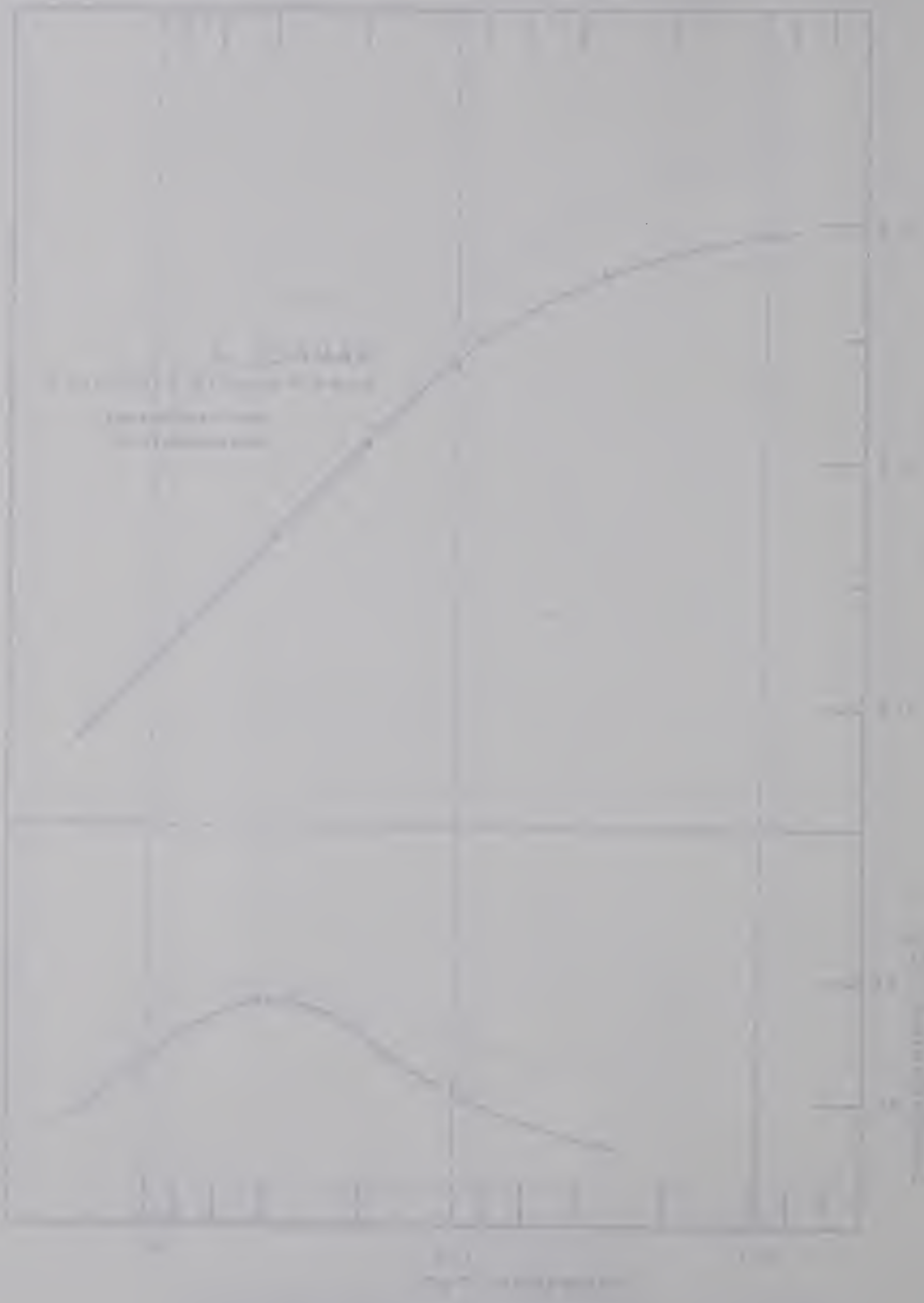


FIGURE 4-2 SAMPLE 4 FORT FRANCIS SILT-CLAY
CONSOLIDATION TEST RESULTS



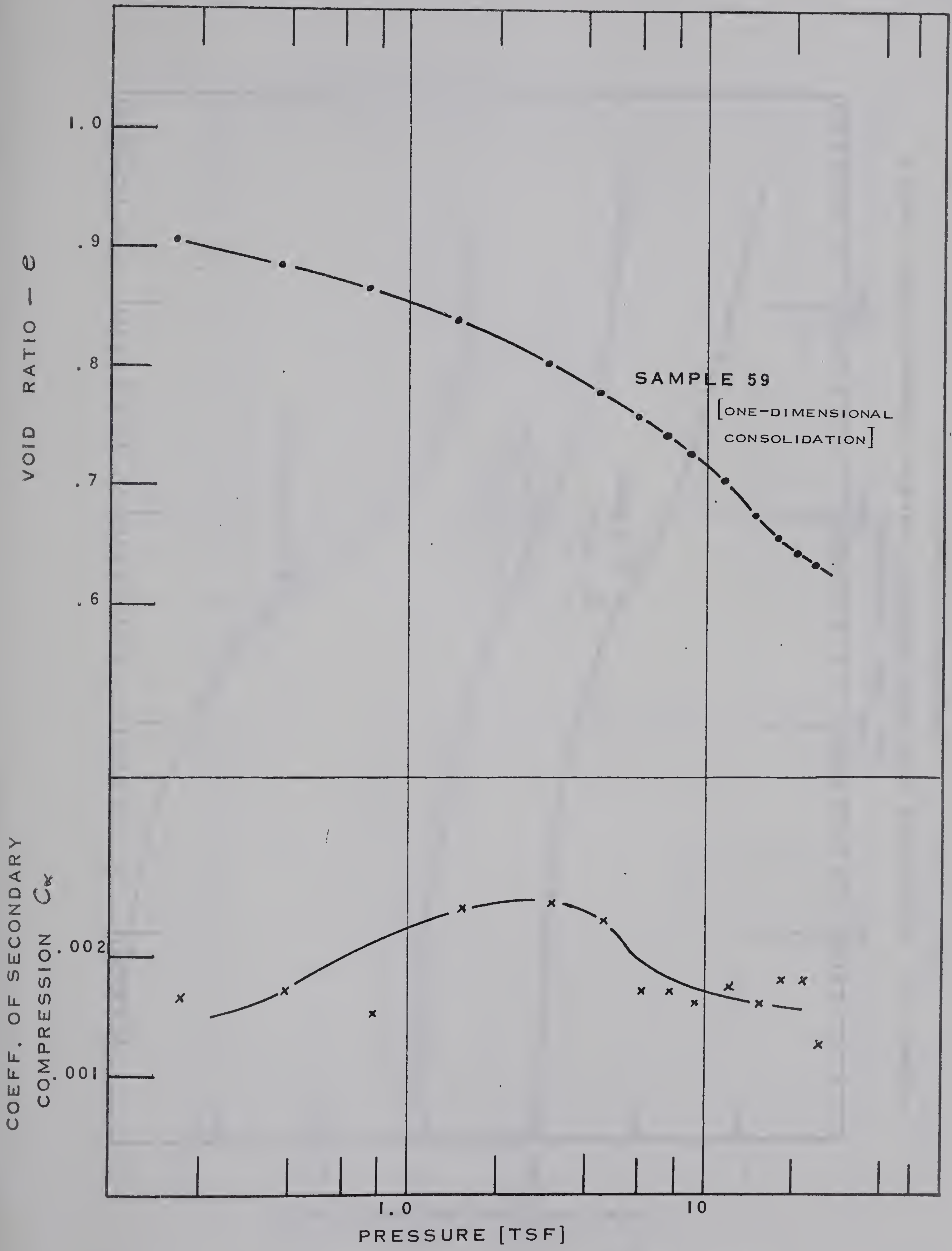


FIGURE 4-3 DUNCAN LAKE SILT-CLAY CONSOLIDATION TEST RESULTS

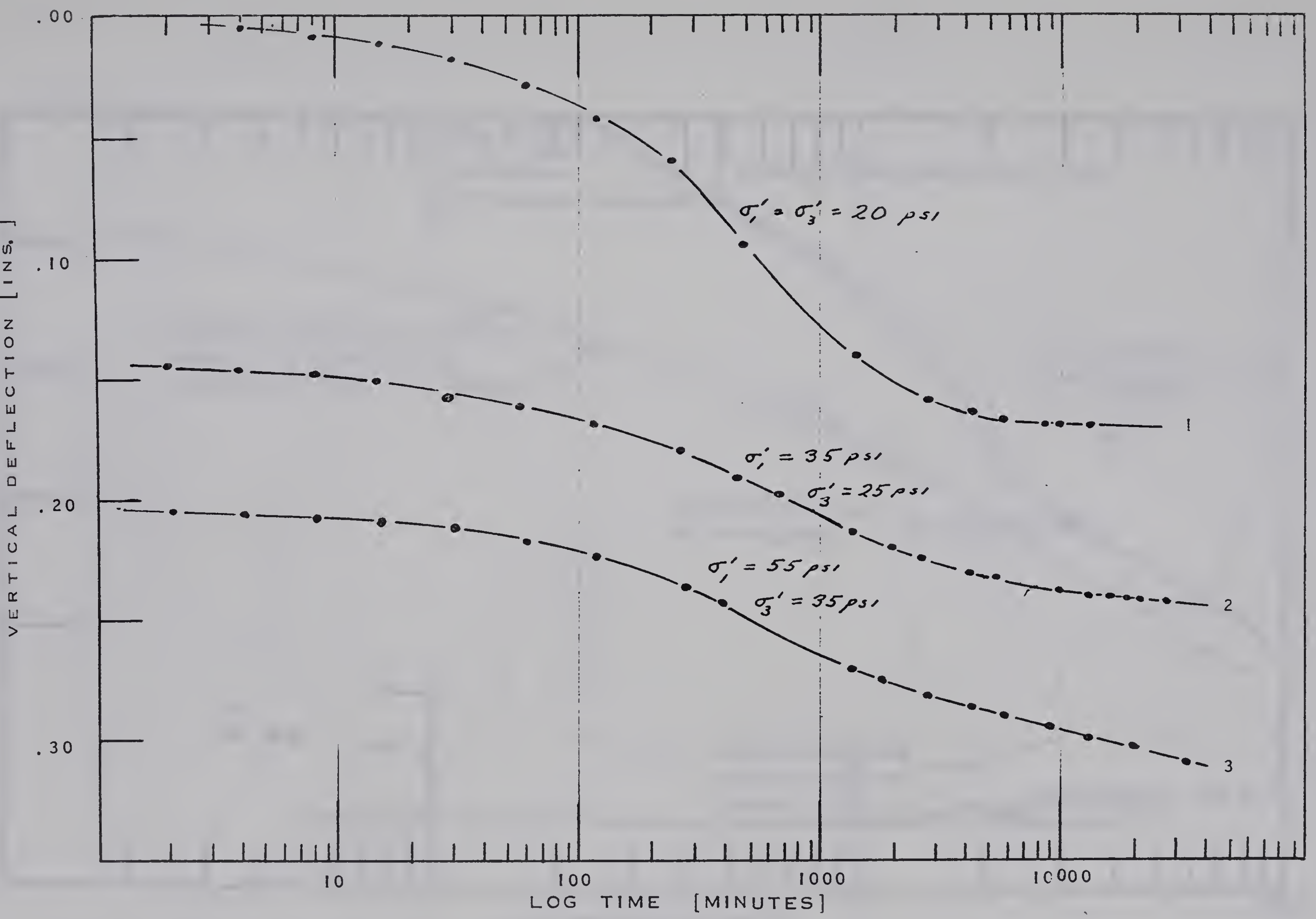
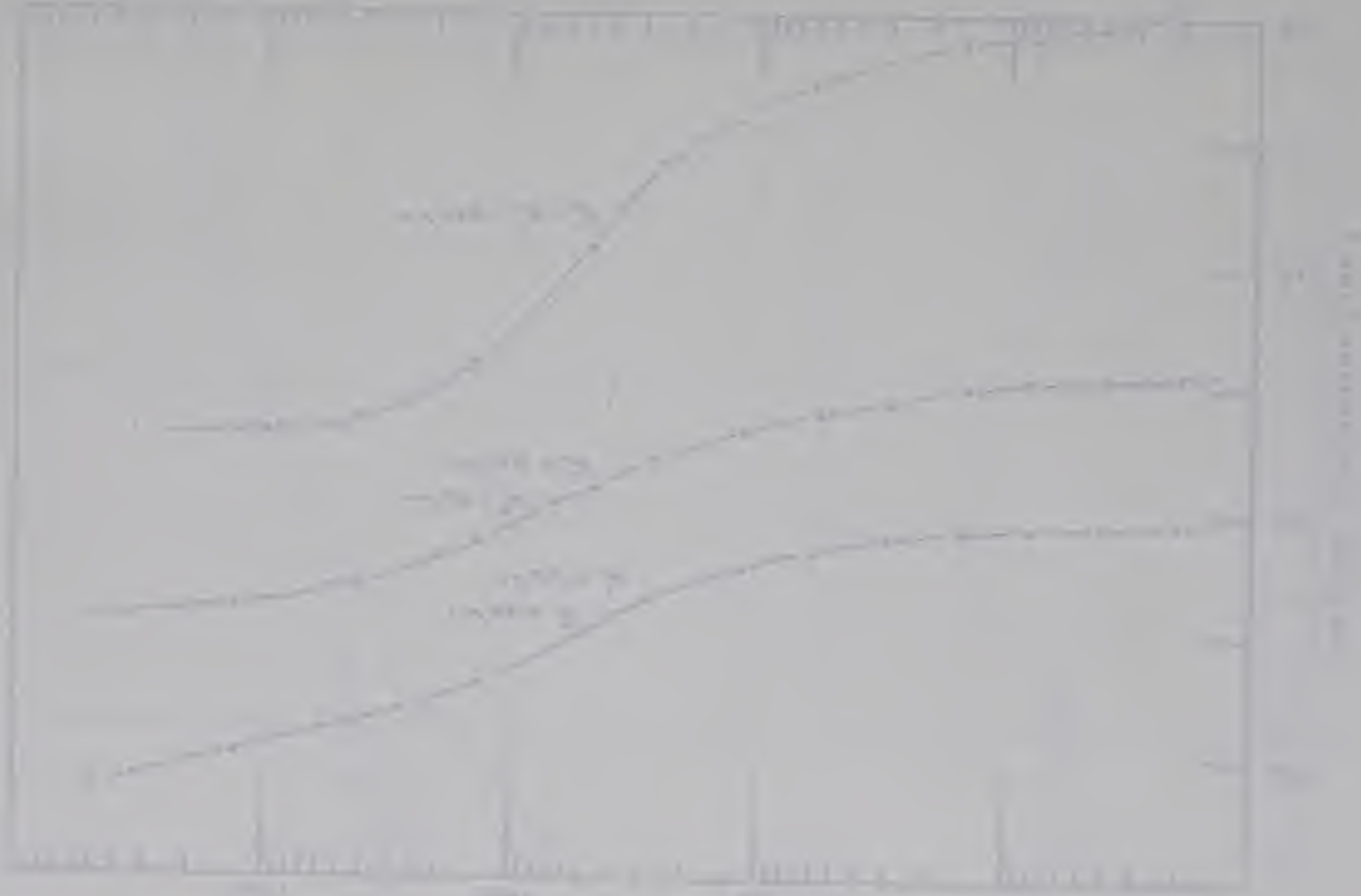


FIGURE 4-4 DEFLECTION LOG TIME CURVES TRIAXIAL CONSOLIDATION TEST-2

FORT FRANCIS SILT-CLAY



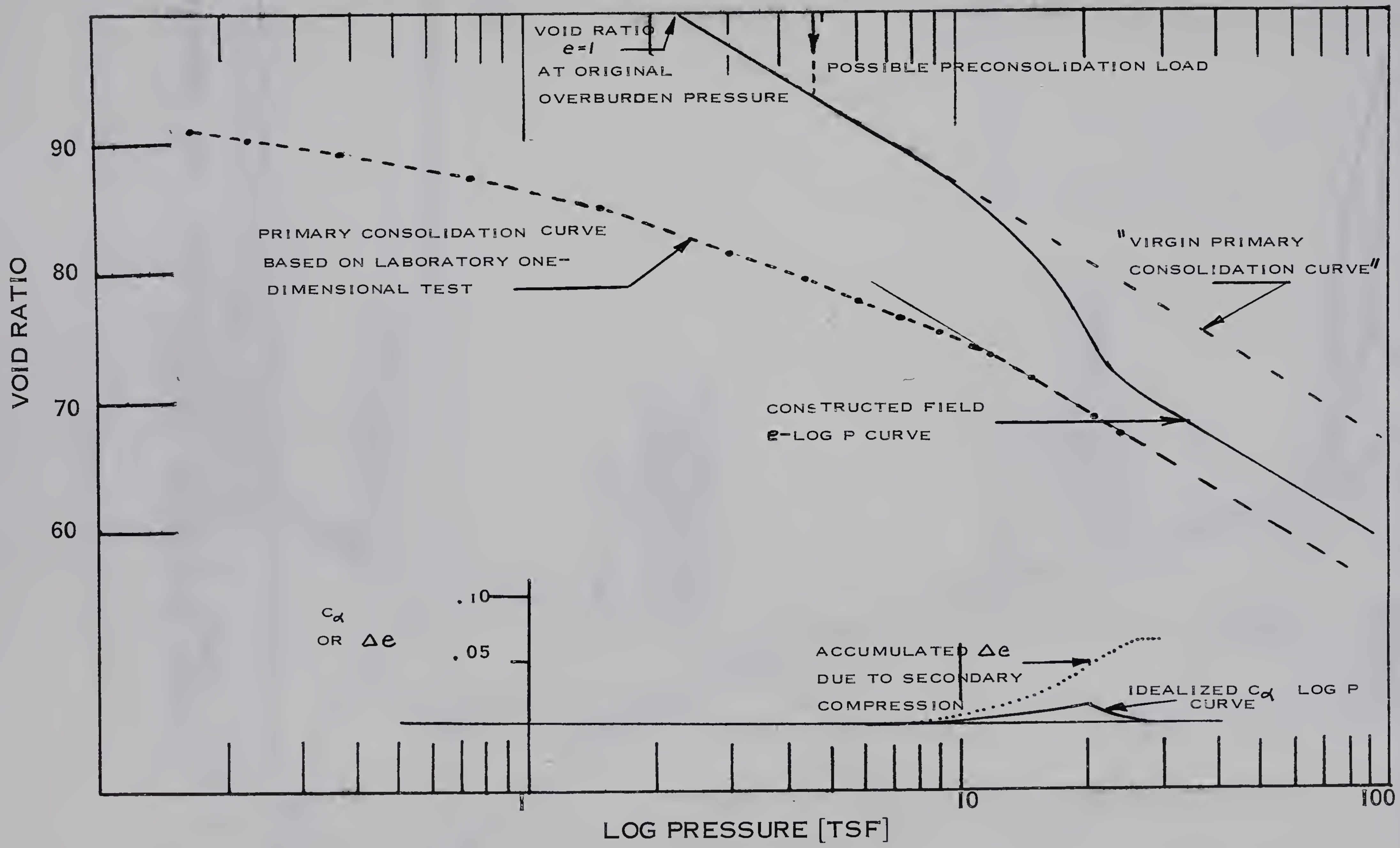


FIGURE 4-5 CONSTRUCTED FIELD LOG P CURVE DUNCAN LAKE SILT-CLAY

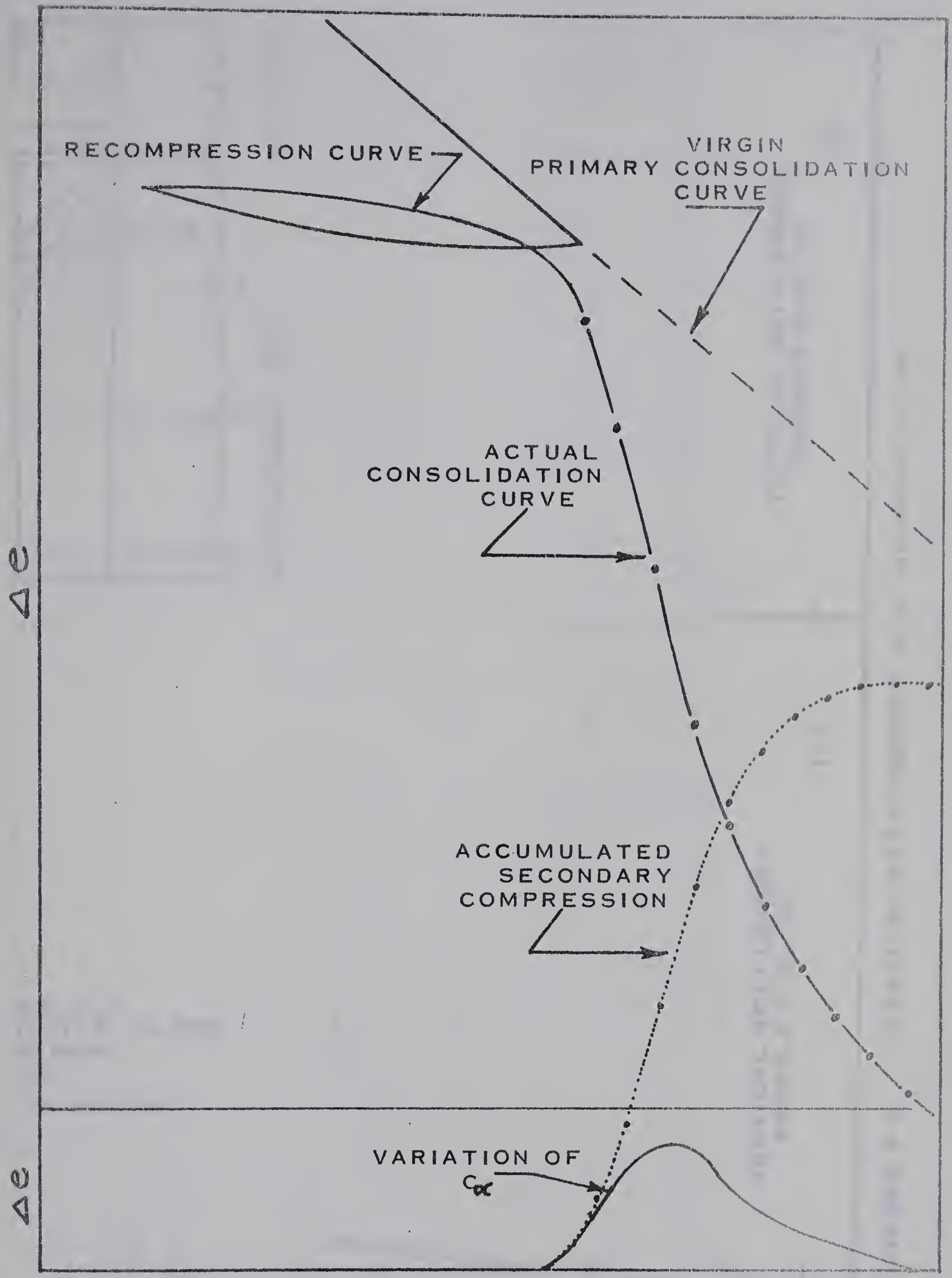


FIGURE 4-6 CONSTRUCTED e-LOG P CURVE FOR A HIGHLY SENSITIVE SOIL

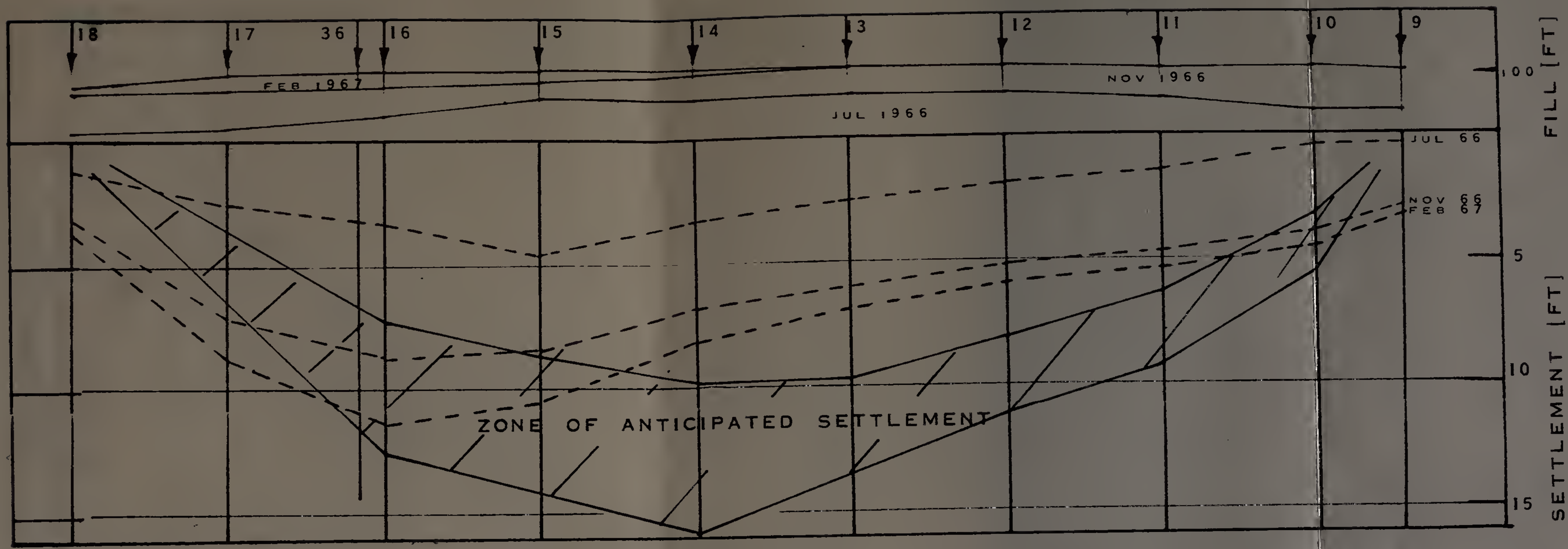


FIGURE 5-1 SETTLEMENT ALONG DUNCAN DAM CENTER LINE

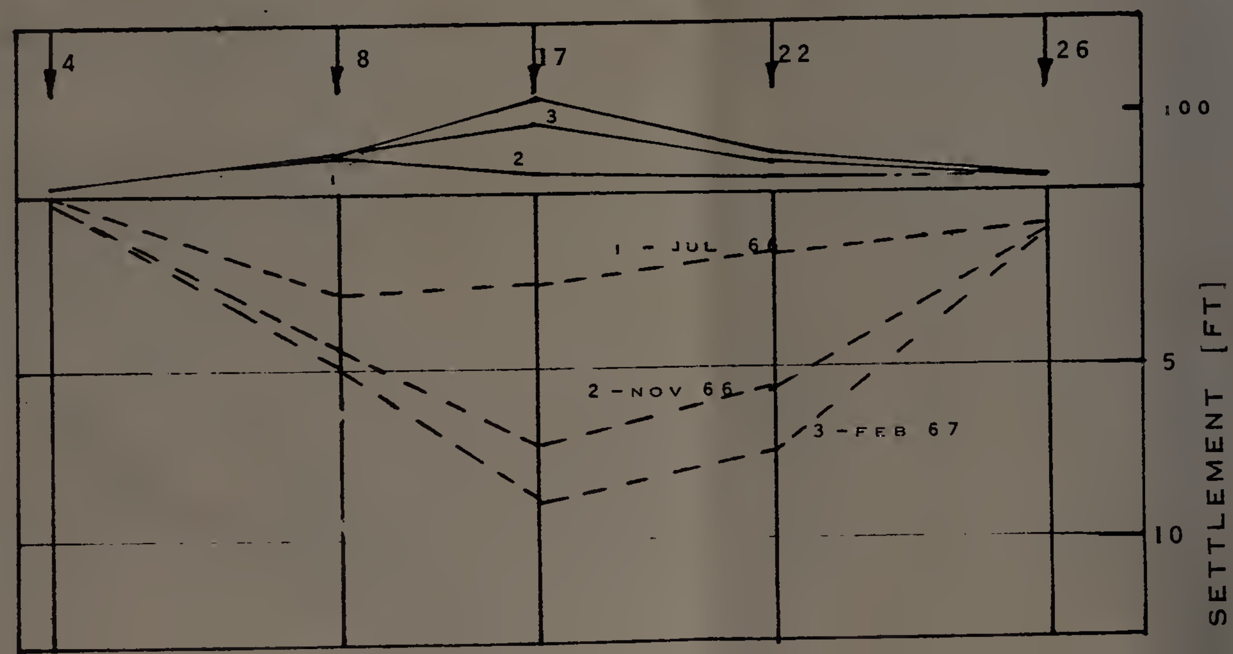


FIGURE 5-2
SETTLEMENT PROFILE ALONG SECTION OF MAXIMUM SETTLEMENT
[GAUGES 4,8,17,22 AND 26]

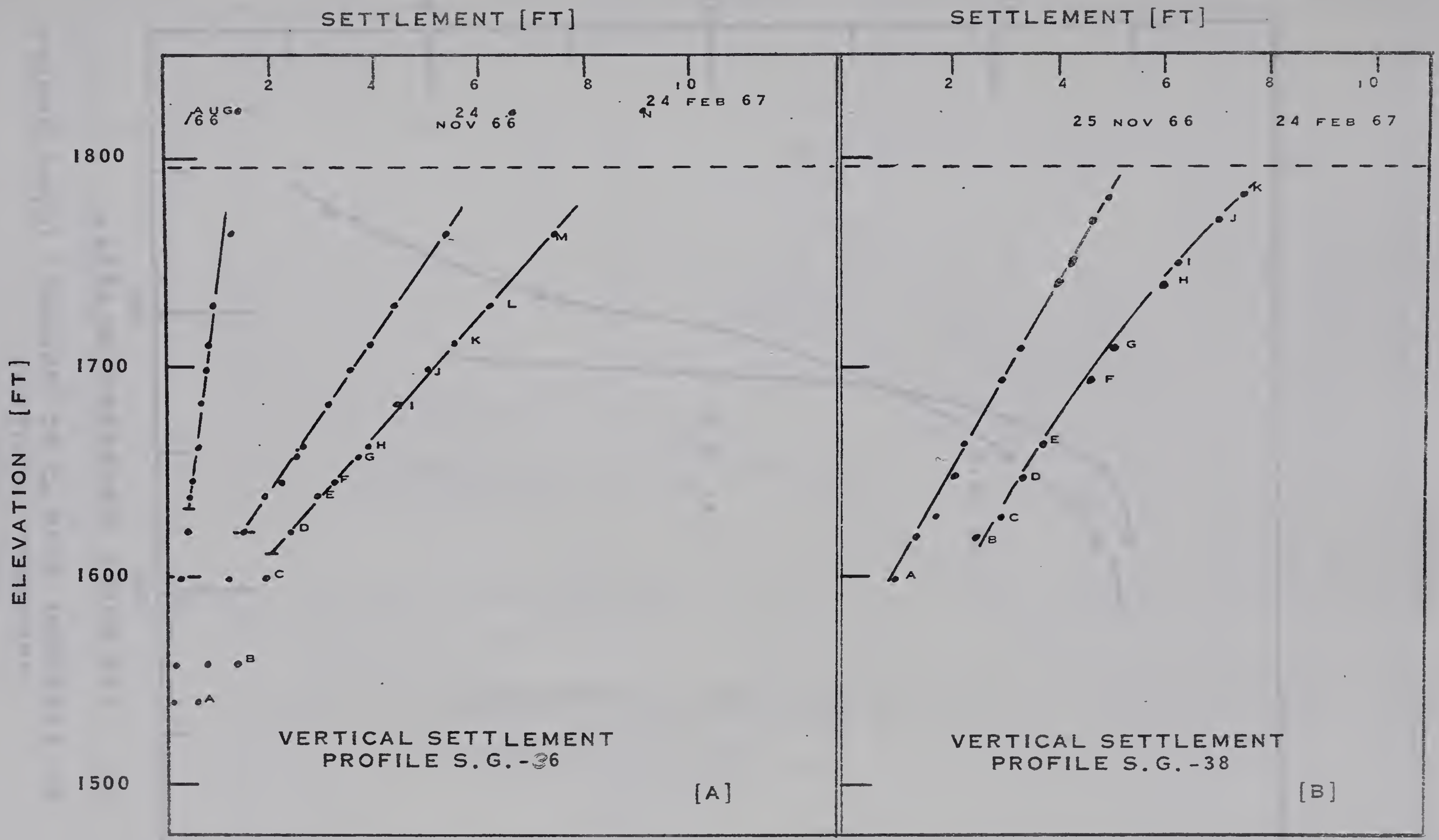
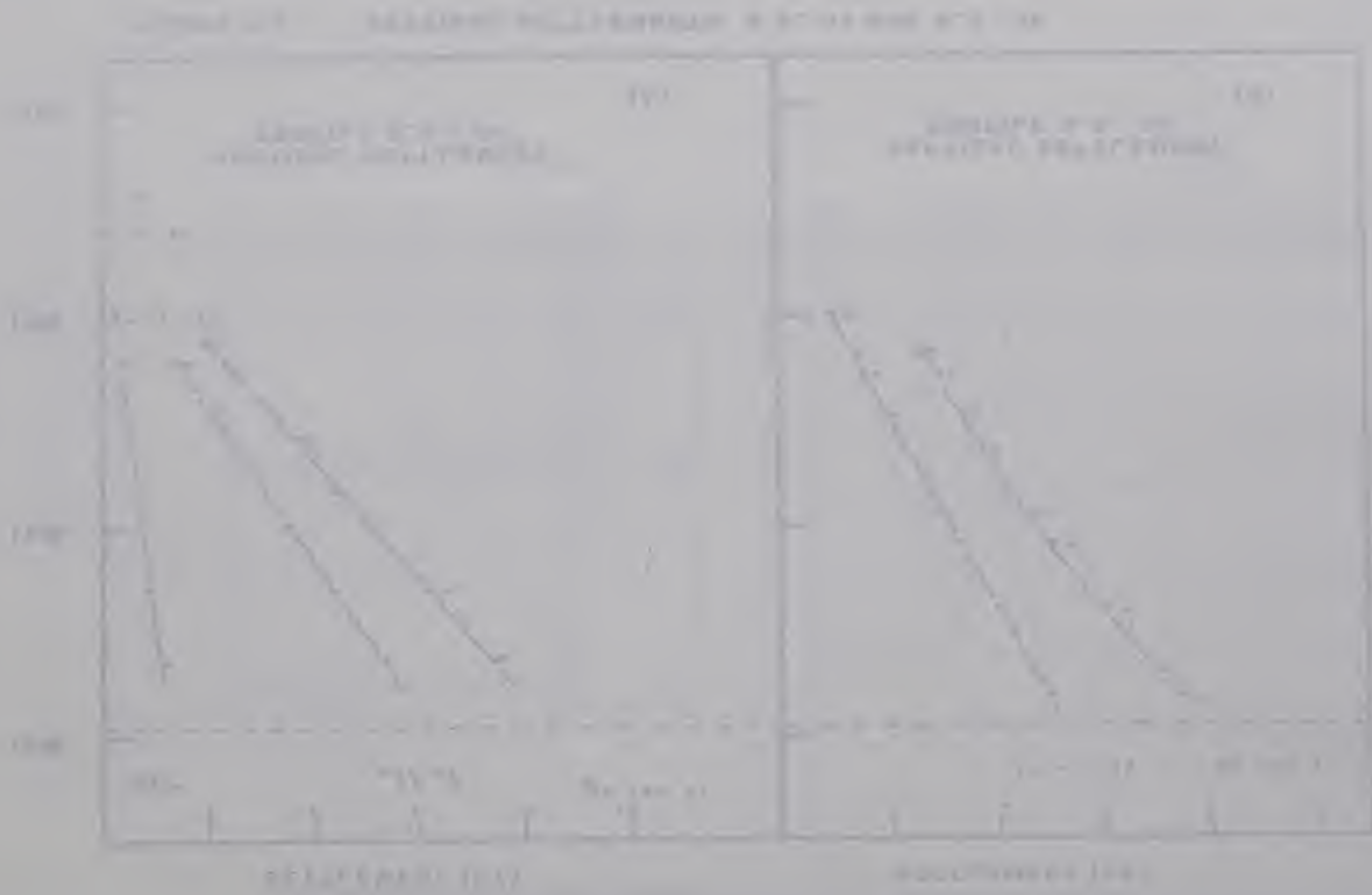


FIGURE 5-3 VERTICAL SETTLEMENTS S.G.-36 AND S.G.-38



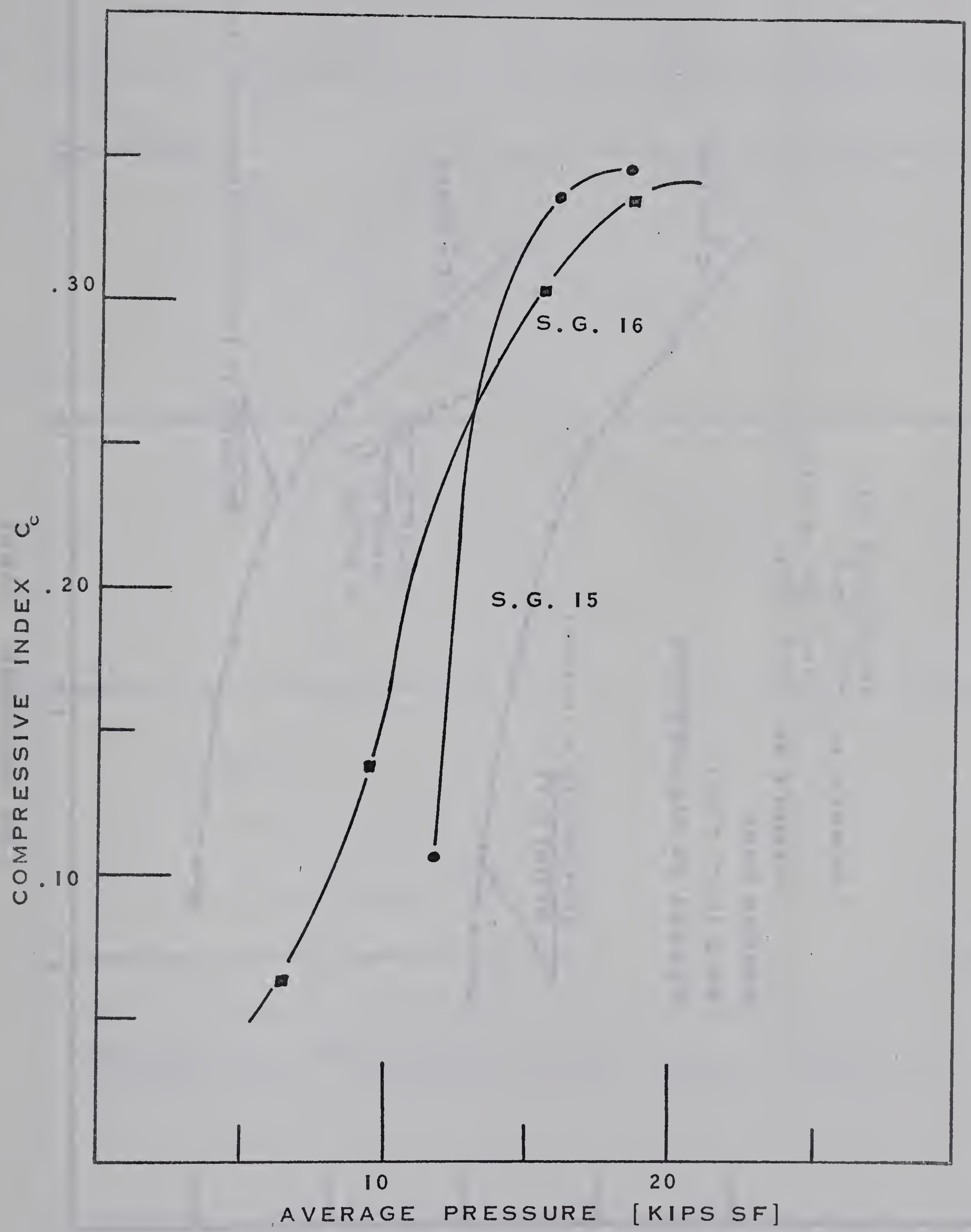


FIGURE 5-3[C] CHANGE OF C_c WITH INCREASE IN VERTICAL PRESSURE

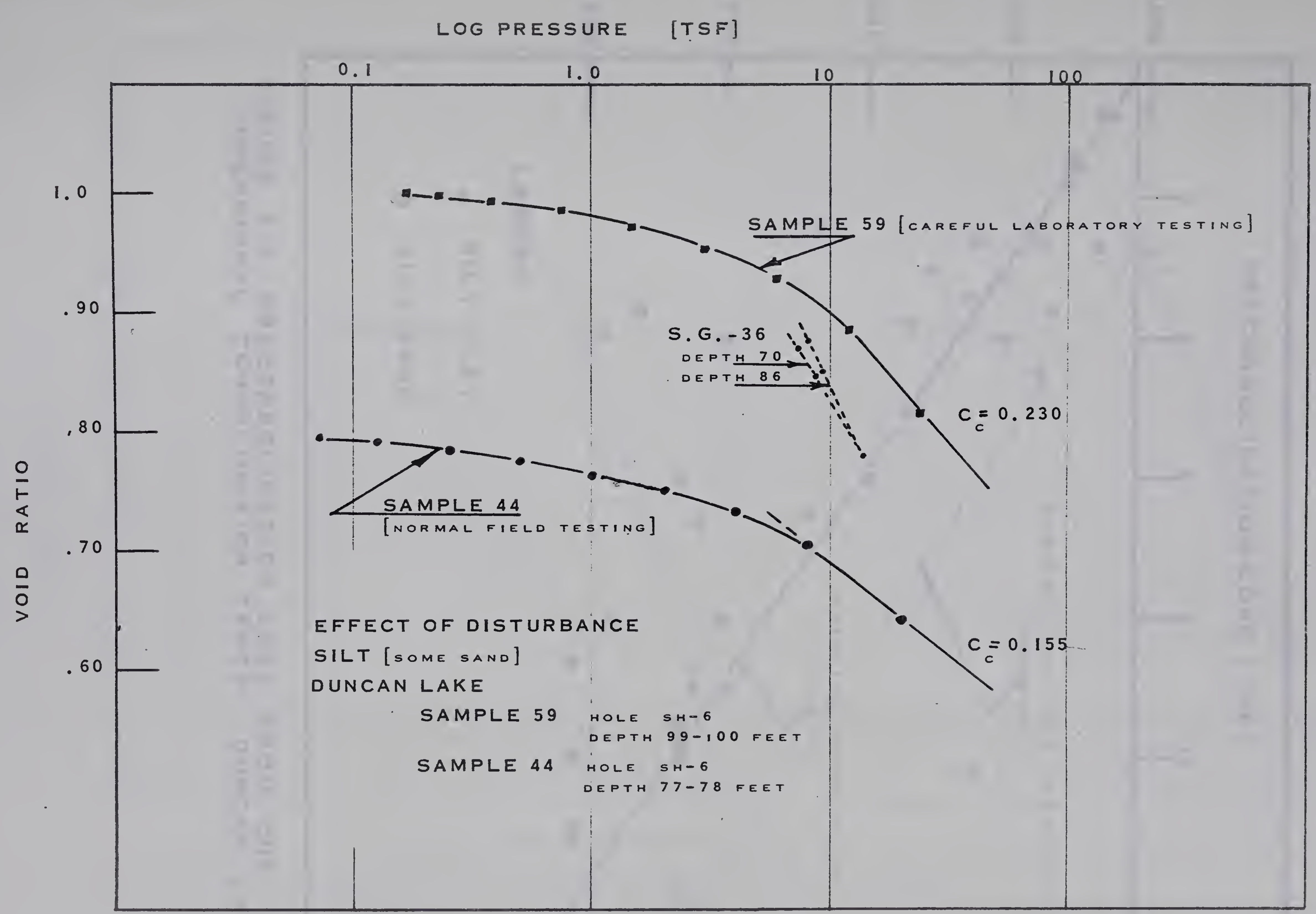


FIGURE 5-4 COMPARISON OF LAB e-LOG P CURVES FOR DUNCAN LAKE SILT

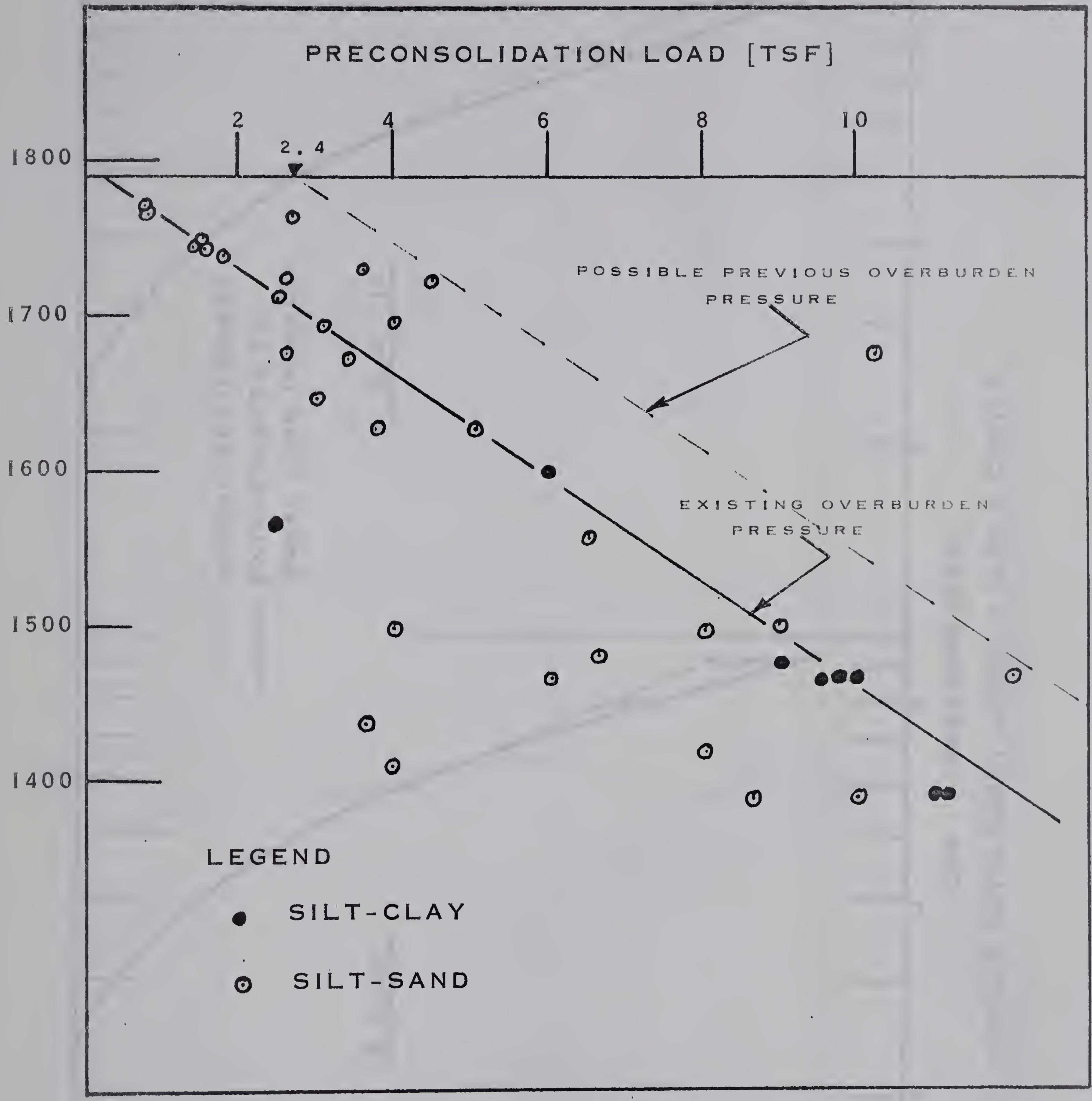


FIGURE 5-5 PRECONSOLIDATION LOAD FROM ONE-DIMENSIONAL CONSOLIDATION TESTS DUNCAN LAKE

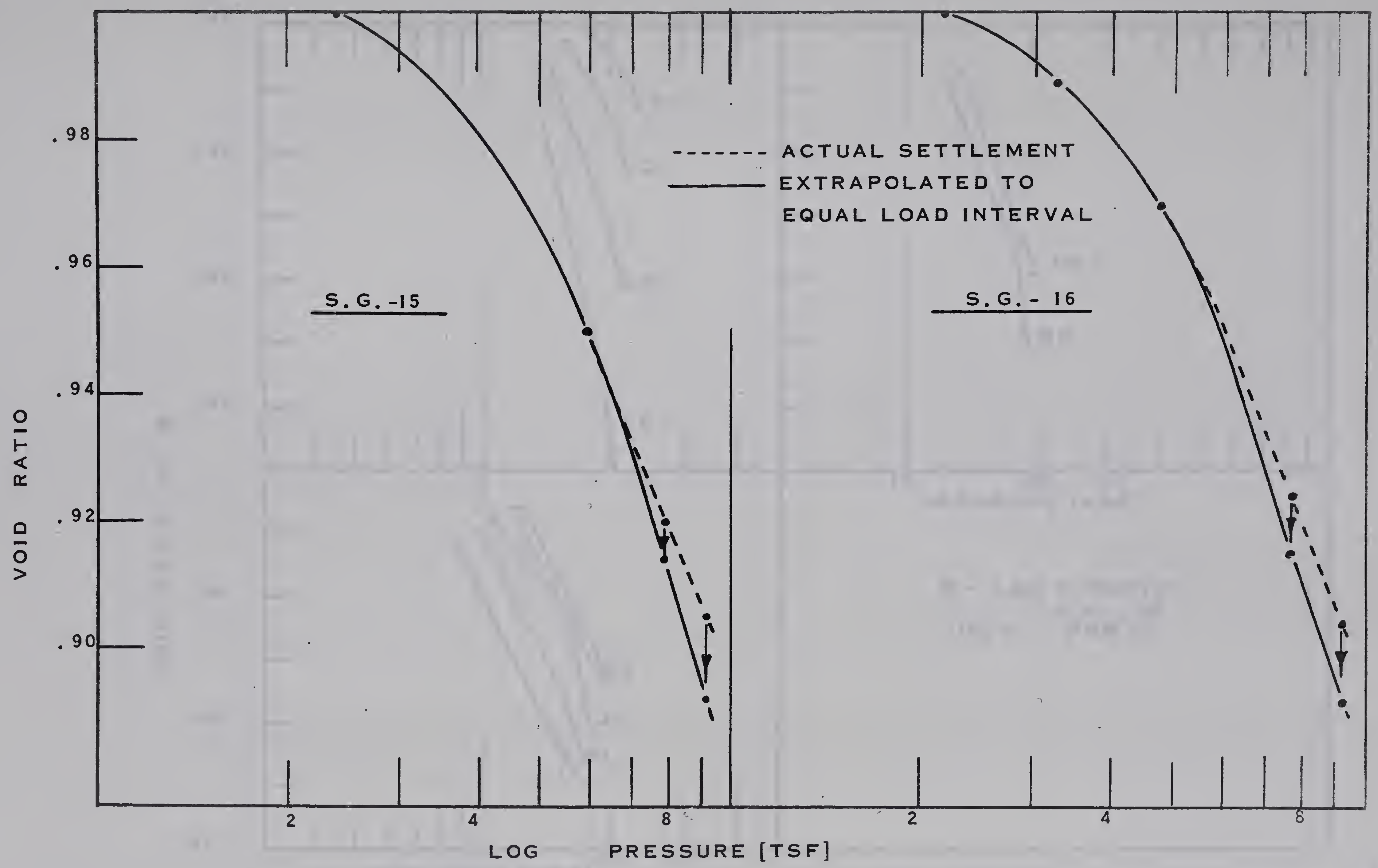
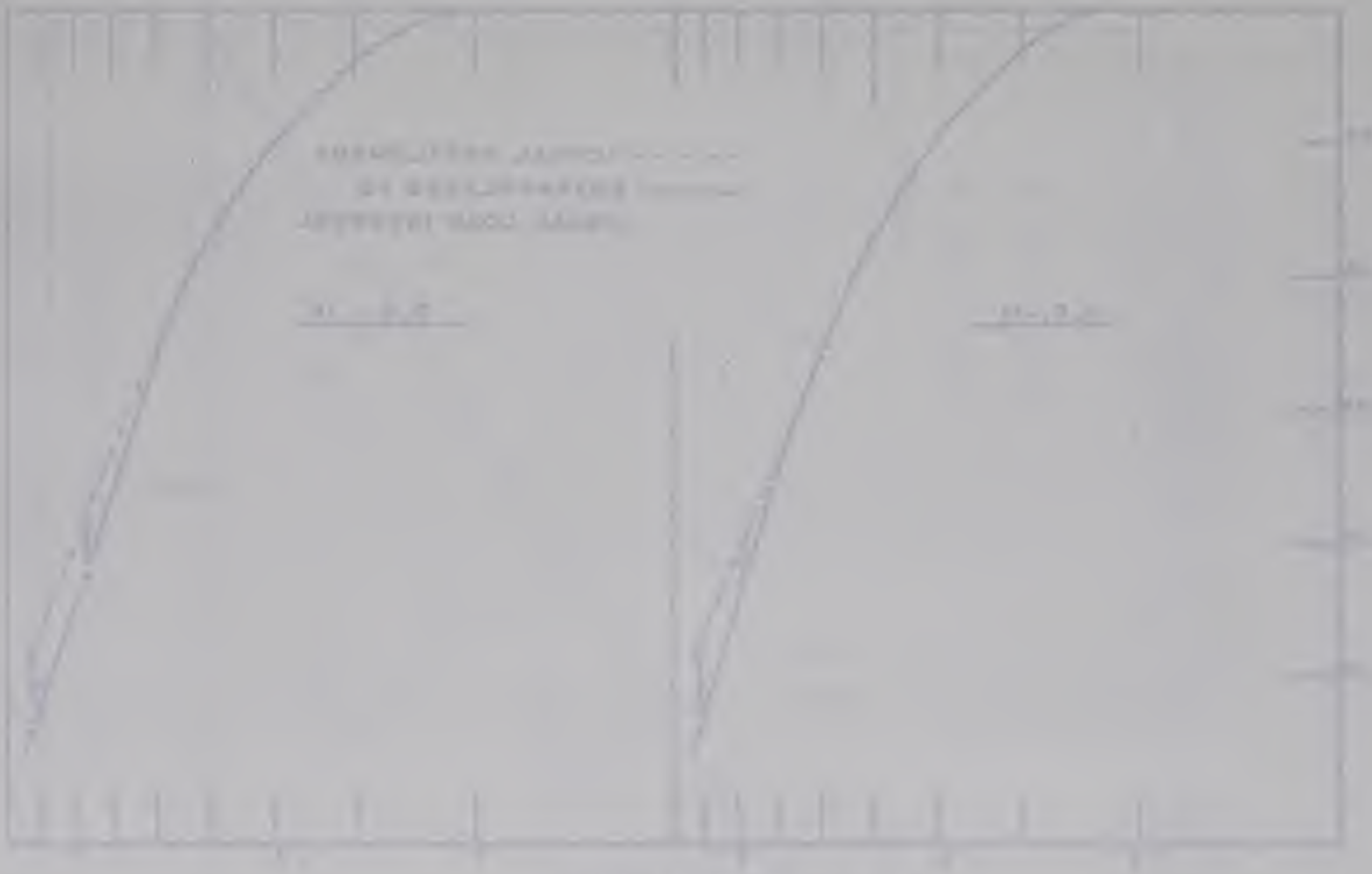


FIGURE 5-6 SURFACE SETTLEMENT GAUGE e-LOG P CURVES



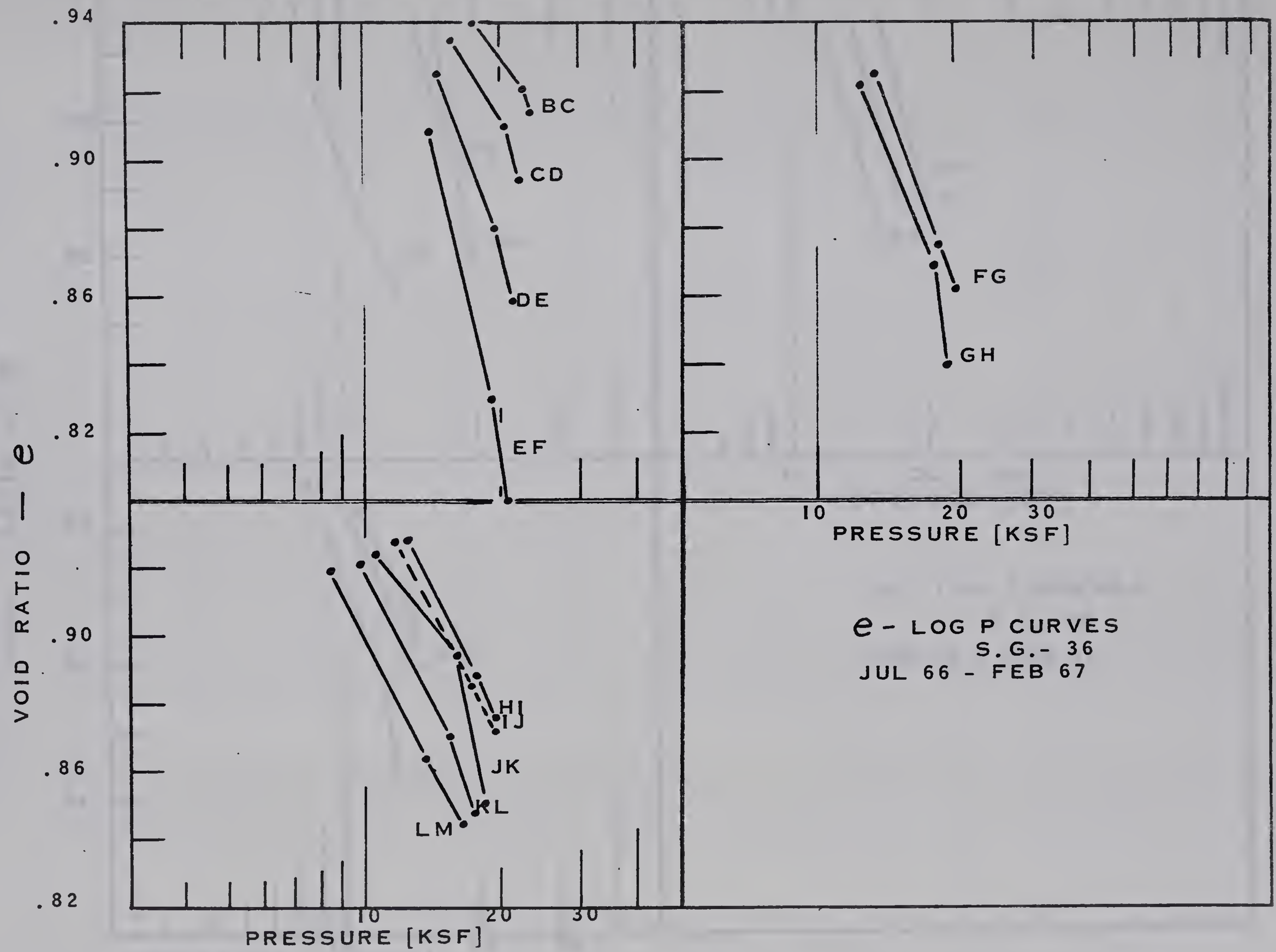
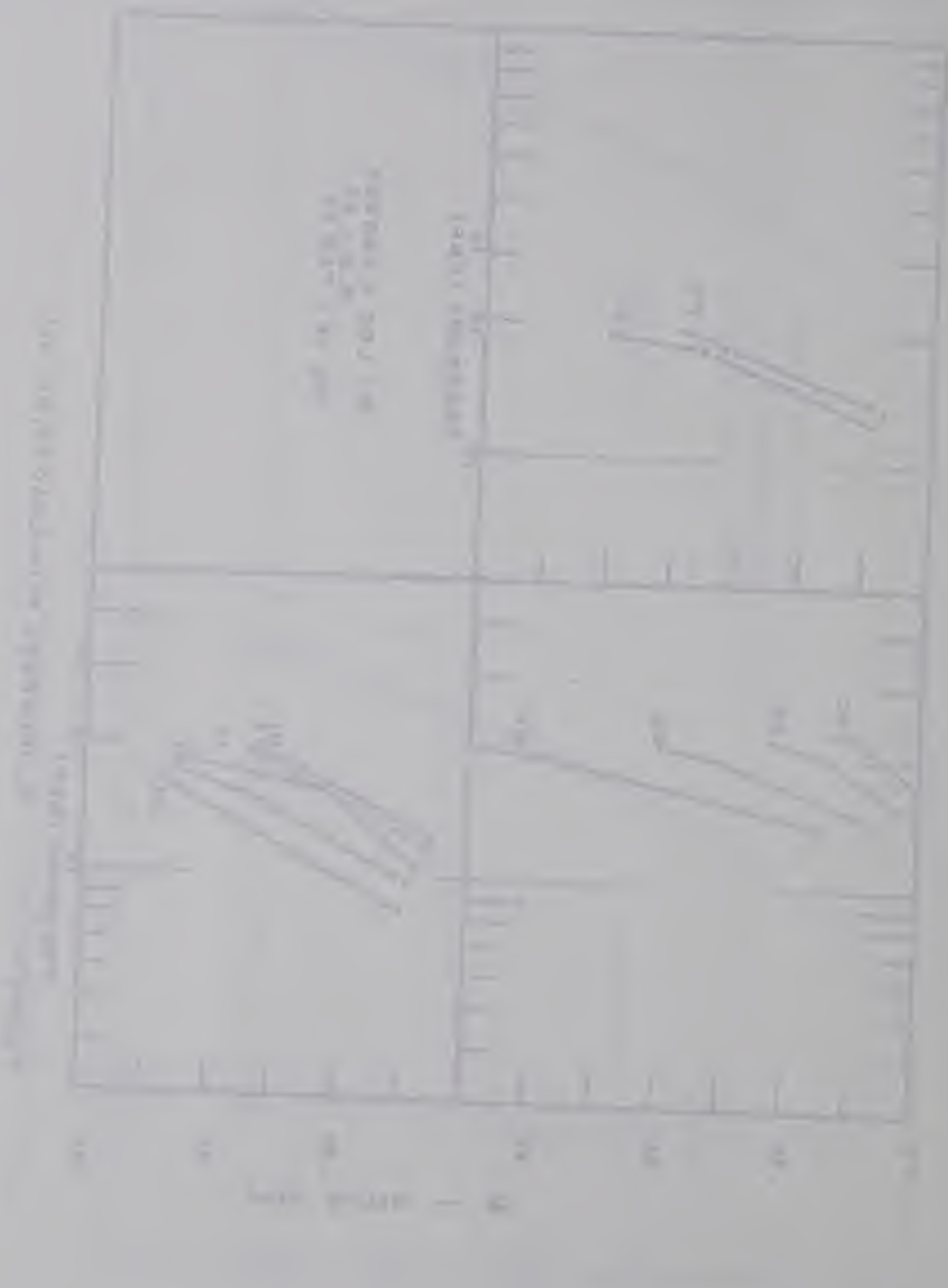


FIGURE 5 - 7 C_c INCREASE WITH LOAD [S.G. - 36]



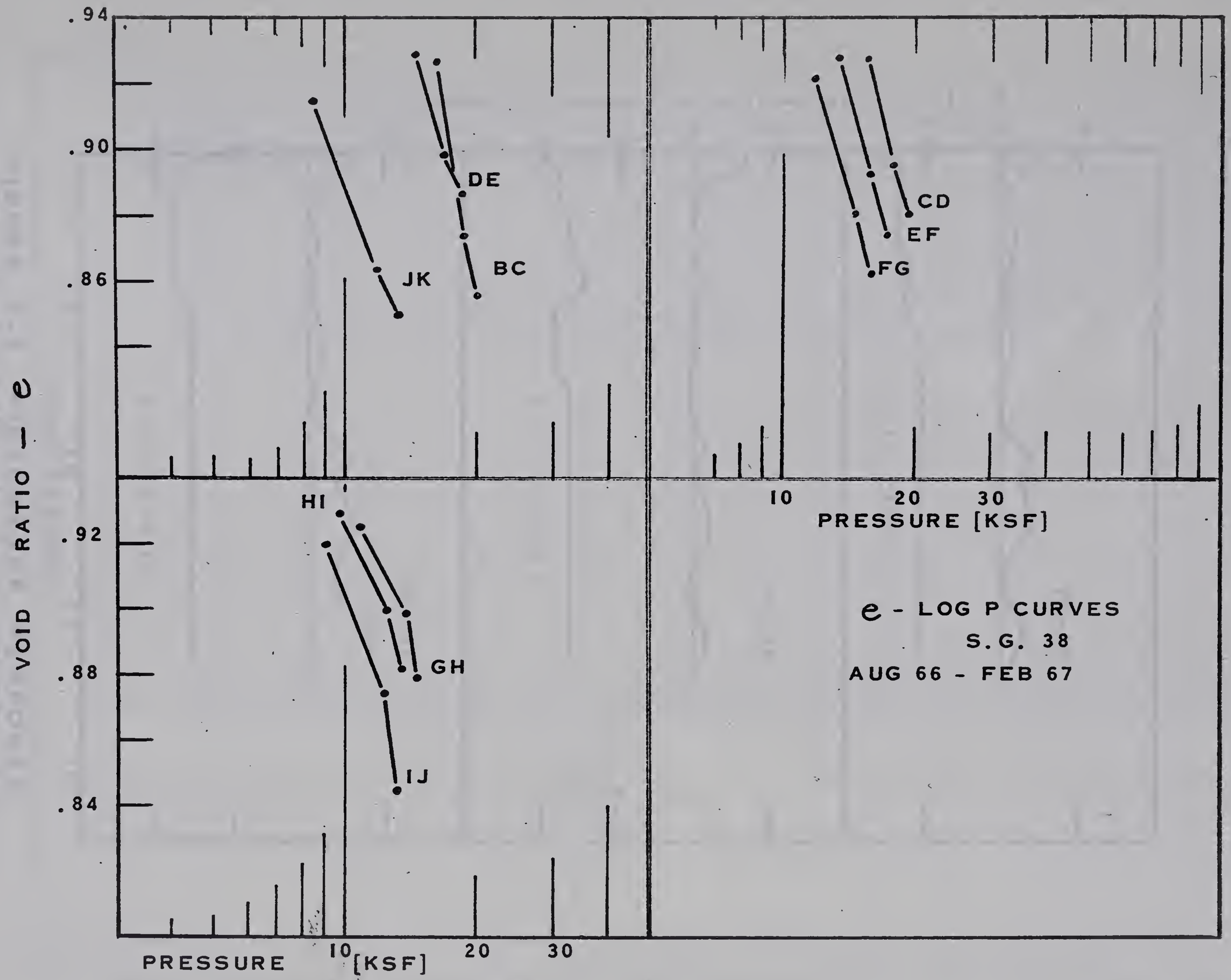
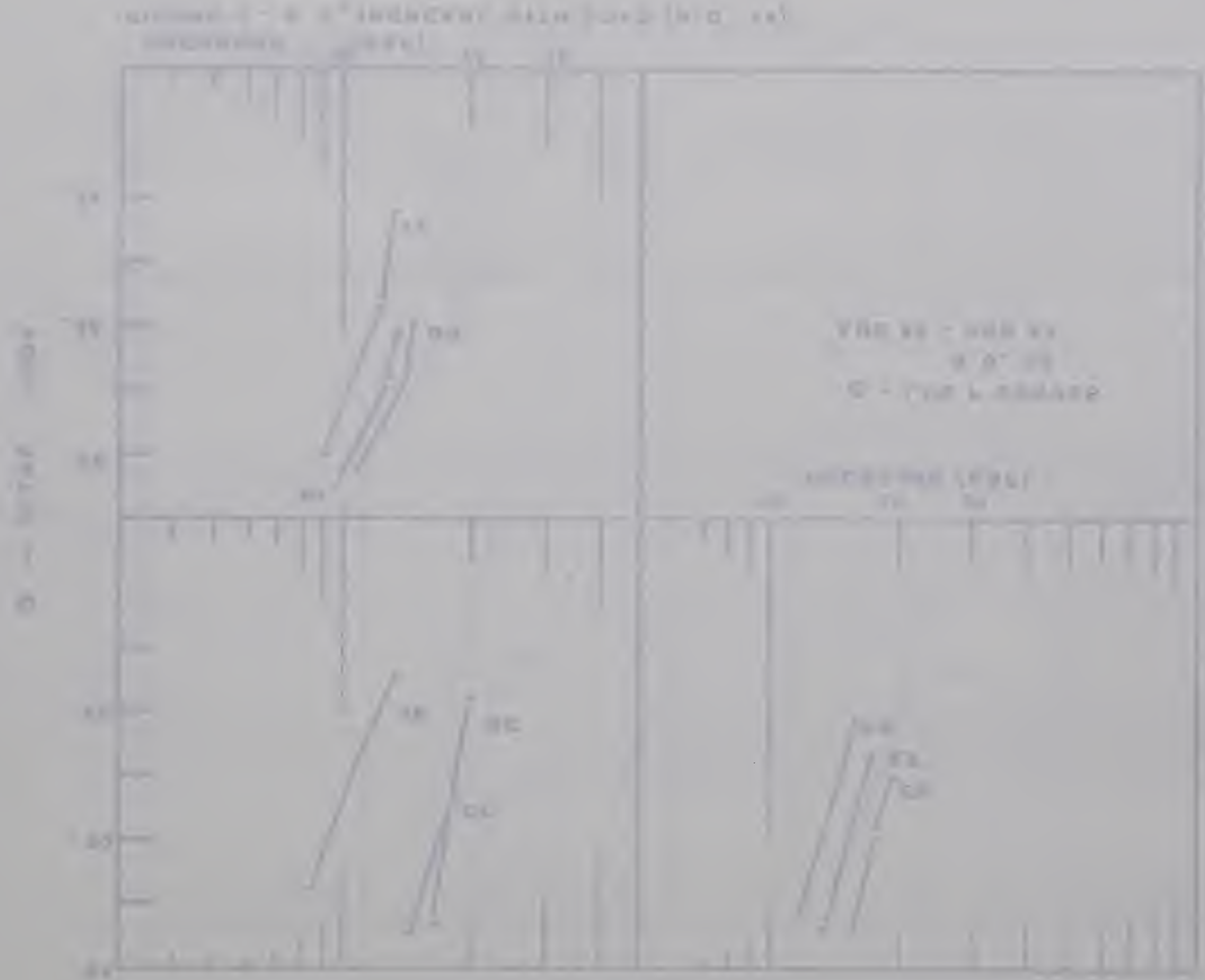


FIGURE 5 - 8 C_c INCREASE WITH LOAD [S.G. 38]



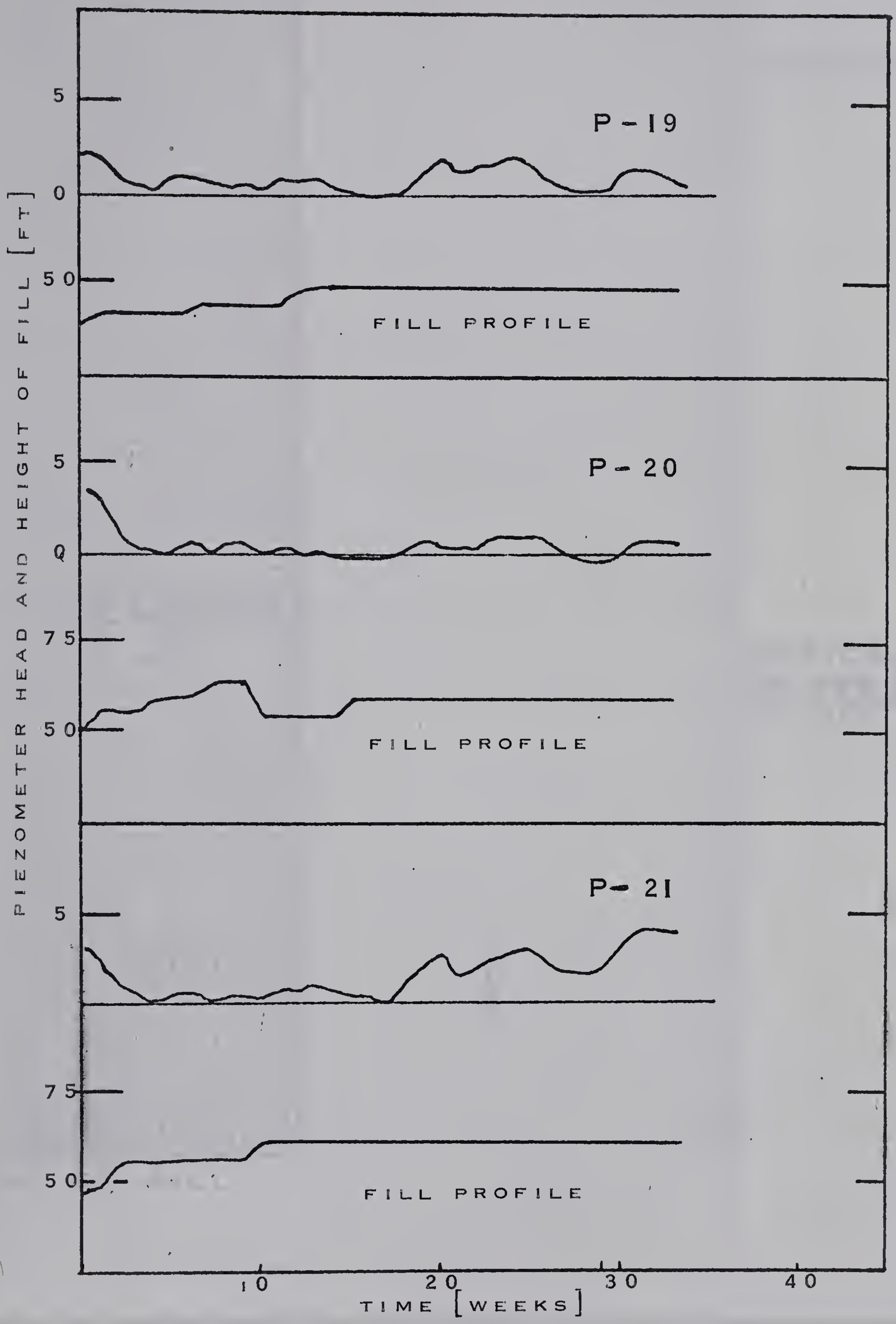


FIGURE 5-9 PIEZOMETER RESPONSE TO LOAD [AUTUMN AND WINTER '65]

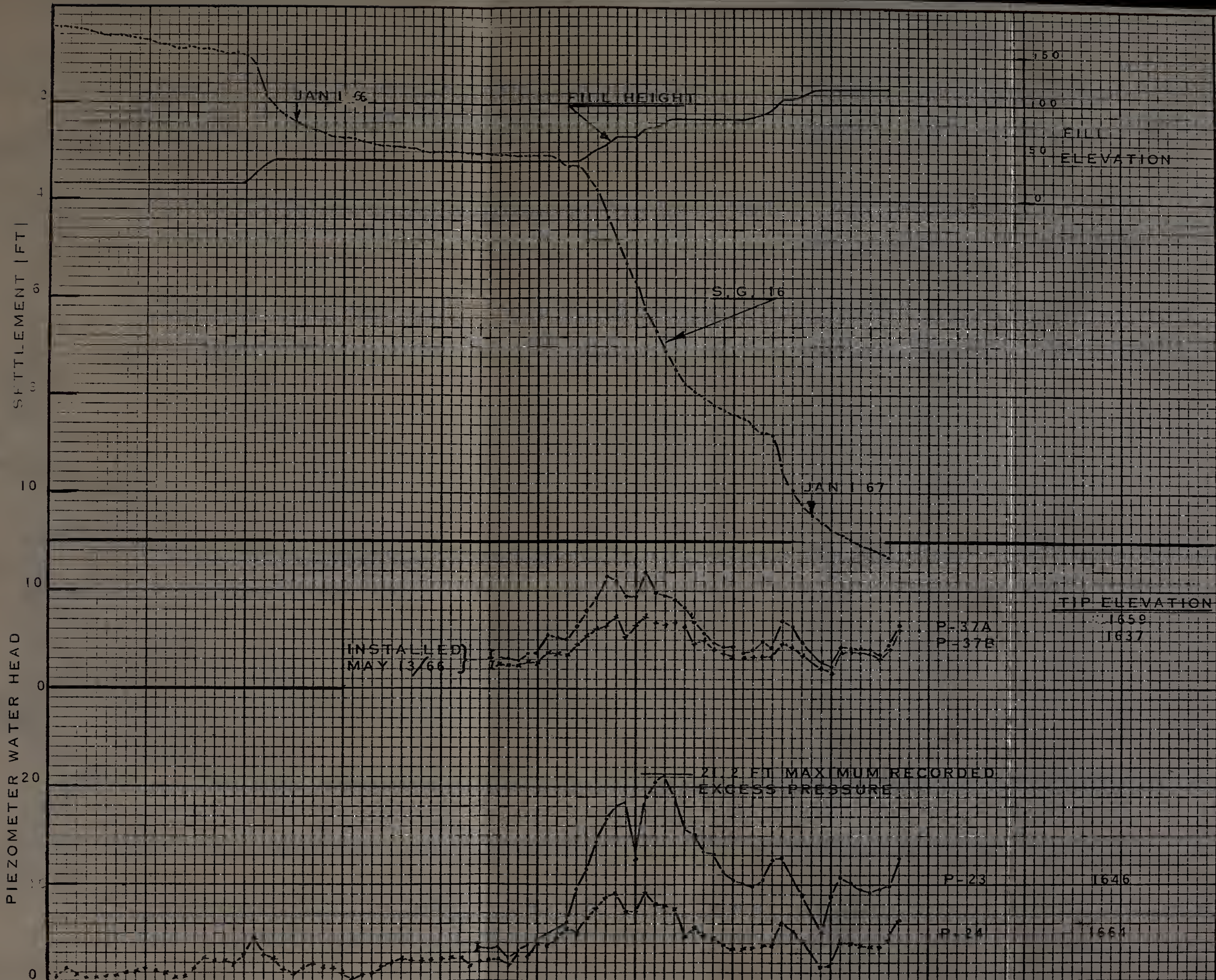


FIGURE 5-10 COMPARISON OF PIEZOMETER RECORDS WITH HEIGHT OF FILL AND SETTLEMENT

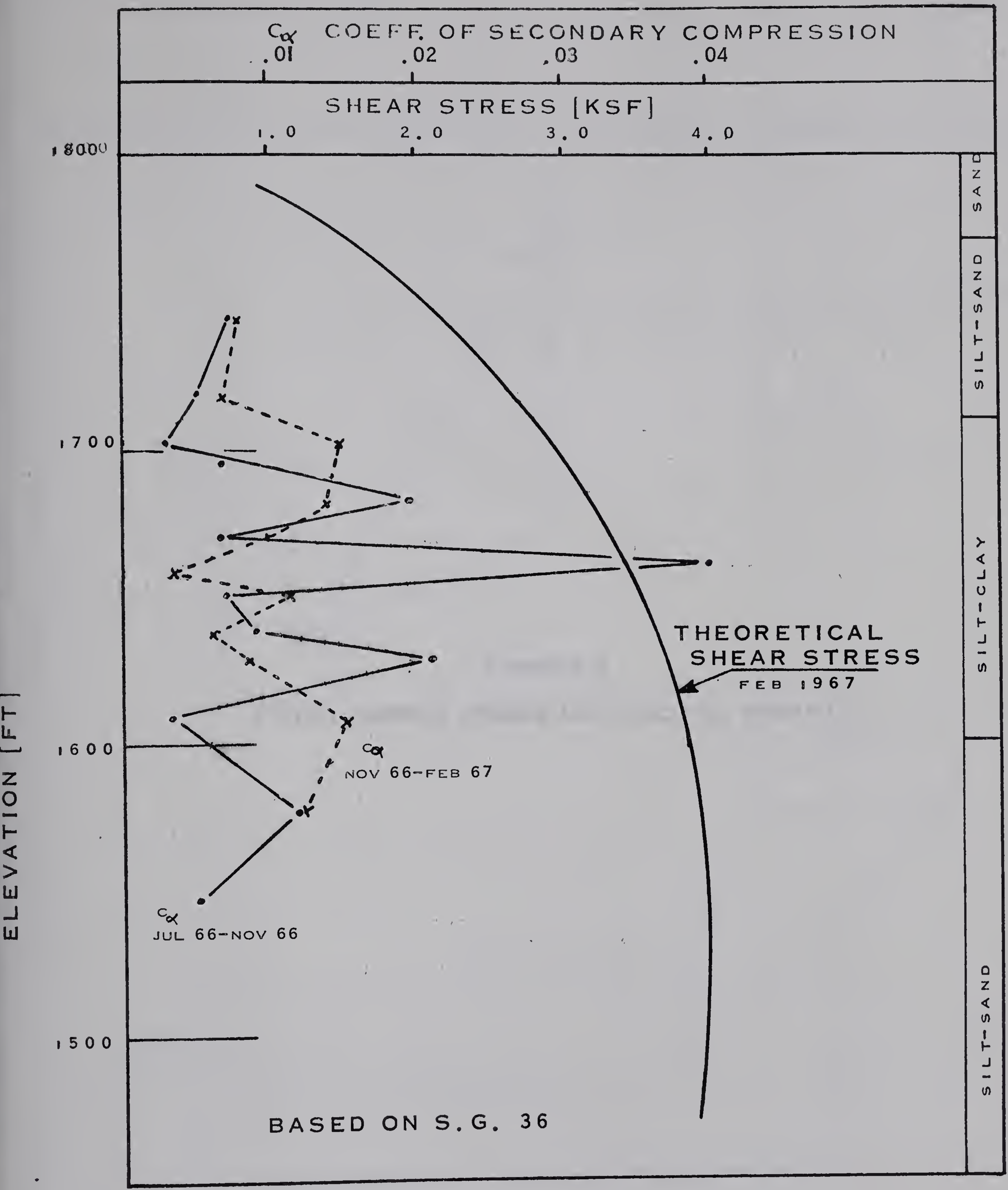


FIGURE 5-11 SHEAR STRESS AND C_α VARIATION WITH DEPTH

Appendix B

DIGITAL COMPUTER PROGRAM AND ASSOCIATED MATERIAL

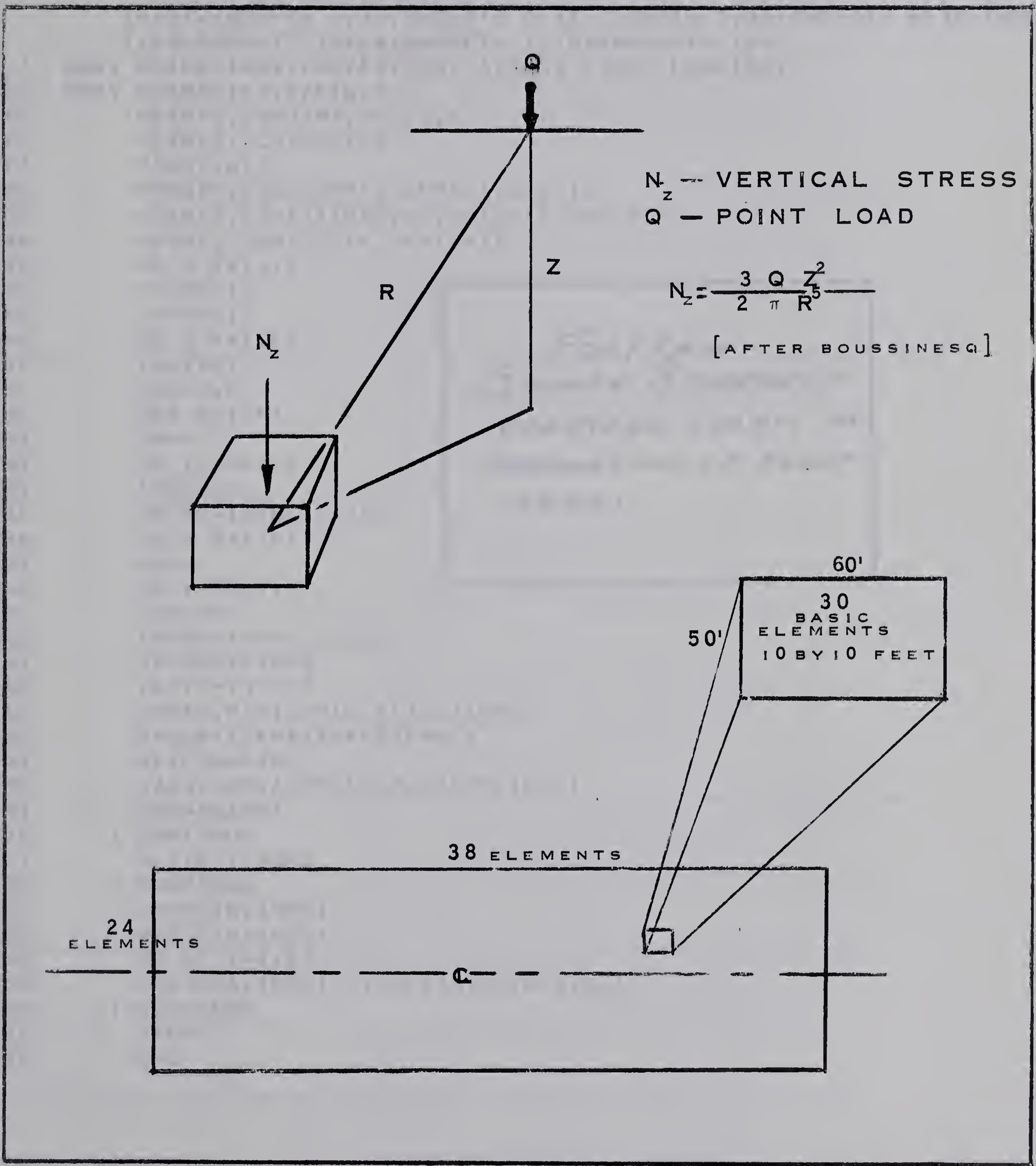


FIGURE B-1 SCHEMATIC VIEW OF VERTICAL STRESS ANALYSIS

FORTRAN
Source Statement
VERTICAL STRESS BY
SUMMATION OF POINT
LOADS

```
$IBFTC ONE      NODECK,NOLIST
      REAL NZ
      DIMENSION PLZ(12,30),Z(12),MP(30),NP(30),D(24,38)
 1000 FORMAT(20I4)
 1001 FORMAT(8F10.4)
 1002 FORMAT(4X,I3,4X,12F10.2)
 1003 FORMAT(4X,3HSTN,6X,7HDEPTH 1,3X,7HDEPTH 2,3X,7HDEPTH 3,3X,7HDEPTH
     14,3X,7HDEPTH 5,3X,7HDEPTH 6,3X,7HDEPTH 7,3X,7HDEPTH 8,3X,7HDEPTH 9
     1,2X,8HDEPTH 10,2X,8HDEPTH 11,2X,8HDEPTH 12)
 1004 FORMAT(40X,29HVERTICAL STRESS FINAL LOADING)
 1005 FORMAT(6X,12F10.4)
      READ(5,1000)MT,NT,IT,KT
      READ(5,1001)DX,DY
      PI=3.1417
      READ(5,1000)(MP(I),NP(I),I=1,IT)
      READ(5,1001)((D(N,M),N=1,NT),M=1,MT)
      READ(5,1001)(Z(K),K=1,KT)
      DO 2 I=1,IT
      XP=MP(I)
      YP=NP(I)
      DO 2 K=1,KT
      ZK=Z(K)
      SNZ=0.0
      DO1 M=1,MT
      AM=M
      DO 1 MM=1,6
      AMM=MM
      XM=AM+(AMM-1.)/6.
      DO 1 N=1,NT
      AN=N
      DO 1 NN=1,5
      ANN=NN
      YN=AN+(ANN-1.)/5.
      X=(XP-XM)*DX
      Y=(YP-YN)*DY
      Q=D(N,M)*10.*10.*130./1000.
      R=SQRT(ZK**2+Y**2+X**2)
      RFIFTH=R**5
      NZ=3.*Q*(ZK**3)/(2.*PI*RFIFTH)
      SNZ=SNZ+NZ
    1 CONTINUE
      PLZ(K,I)=SNZ
    2 CONTINUE
      WRITE(6,1004)
      WRITE(6,1003)
      DO 11 I=1,IT
      WRITE(6,1002) I,(PLZ(K,I),K=1,KT)
   11 CONTINUE
      STOP
      END
```

JOB STRESS

OBJECT PROGRAM IS BEING ENTERED INTO STORAGE.

VERTICAL STRESS INTERMEDIATE LOADING (DEC 22, 66)

STN	DEPTH 1	DEPTH 2	DEPTH 3	DEPTH 4	DEPTH 5	DEPTH 6	DEPTH 7	DEPTH 8
1 9	12.62	10.28	8.53	7.33	6.44	5.77	4.78	4.08
2 10	13.25	12.02	10.70	9.48	8.41	7.51	6.12	5.11
3 11	13.29	12.27	11.26	10.33	9.49	8.72	7.40	6.31
4 12	13.30	12.29	11.32	10.44	9.66	8.96	7.75	6.74
5 13	13.30	12.29	11.33	10.46	9.70	9.01	7.85	6.88
6 14	13.30	12.29	11.33	10.47	9.70	9.03	7.87	6.92
7 15	13.30	12.29	11.33	10.46	9.70	9.02	7.85	6.89
8 16	13.30	12.30	11.33	10.46	9.68	8.98	7.77	6.76
9 17	13.29	12.28	11.29	10.37	9.52	8.75	7.41	6.32
10 18	12.39	11.45	10.31	9.20	8.21	7.36	6.03	5.06
11 4	1.51	1.54	1.66	1.82	2.00	2.16	2.41	2.56
12 26	1.29	1.35	1.48	1.64	1.80	1.95	2.19	2.35

Appendix C

SAMPLE DATA SHEETS AND PHOTOGRAPHS FROM LABORATORY TEST PROGRAM

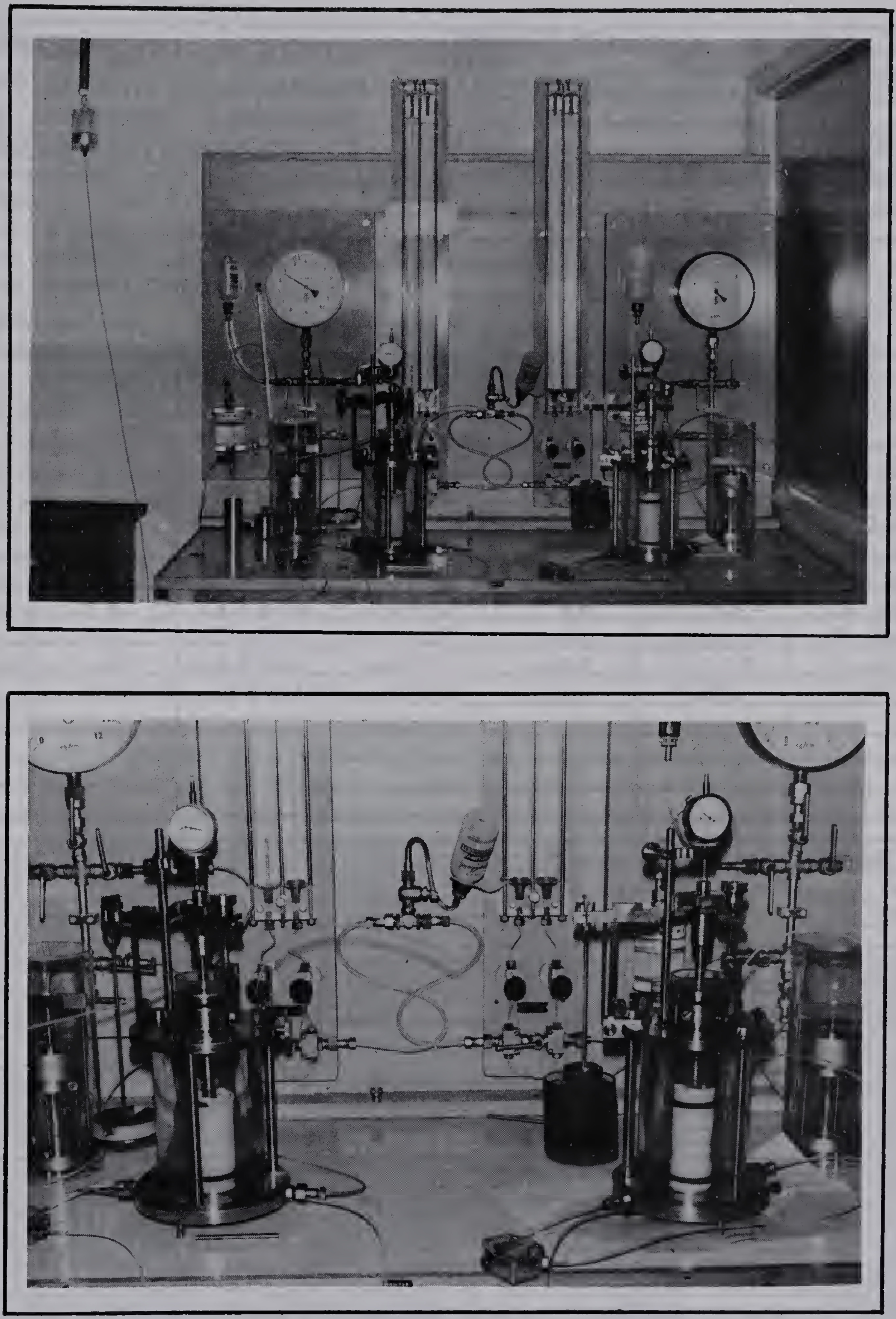


PHOTO C-1 ANISOTROPIC TRIAXIAL CONSOLIDATION TEST APPARATUS

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
CONSOLIDATION RESULTS

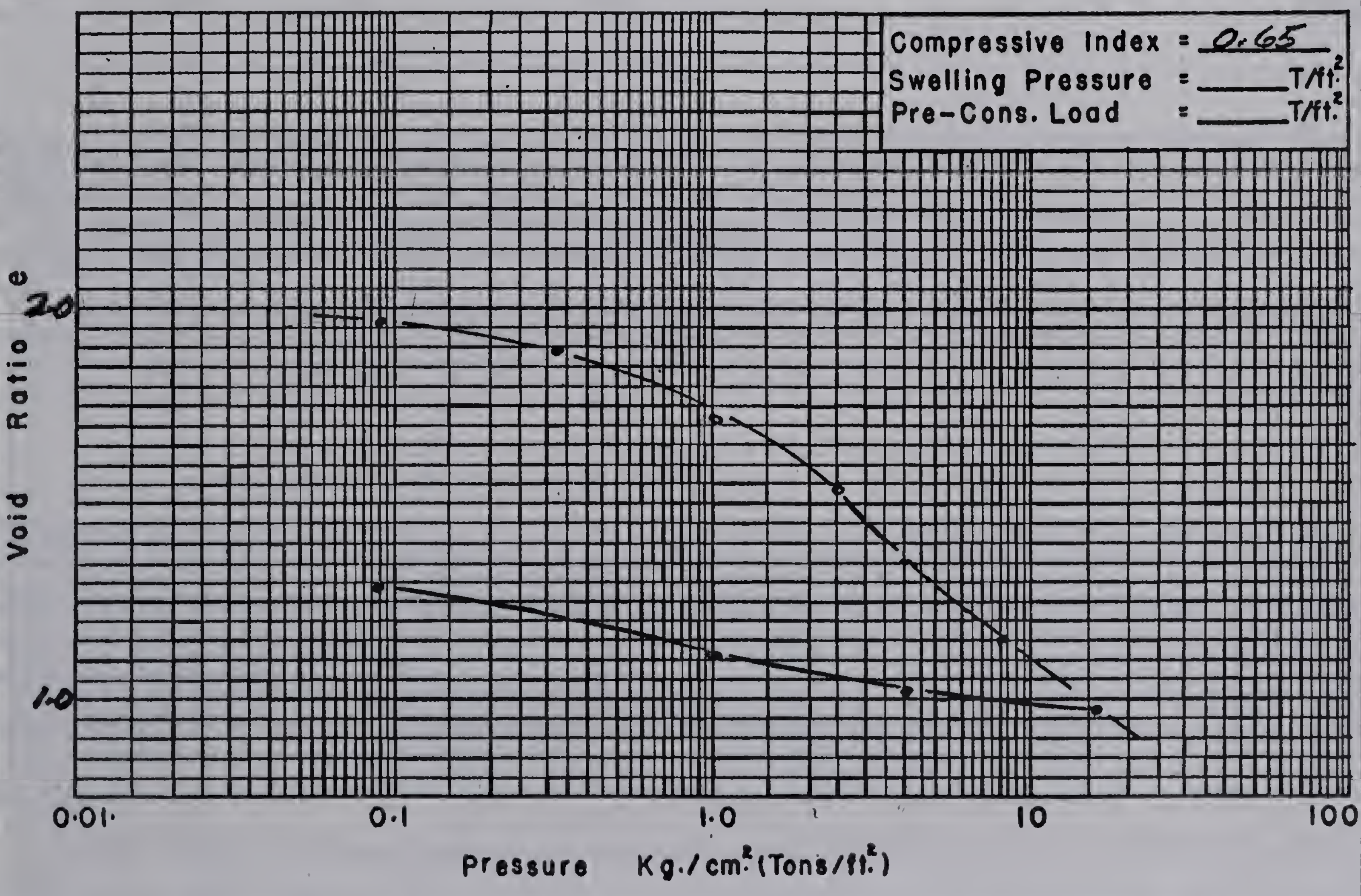
PROJECT THESIS
SITE Fort Francis Ont
SAMPLE Undisturbed (Block
LOCATION CNR Line
HOLE DEPTH ≈12
TECHNICIAN CFH DATE Feb 67

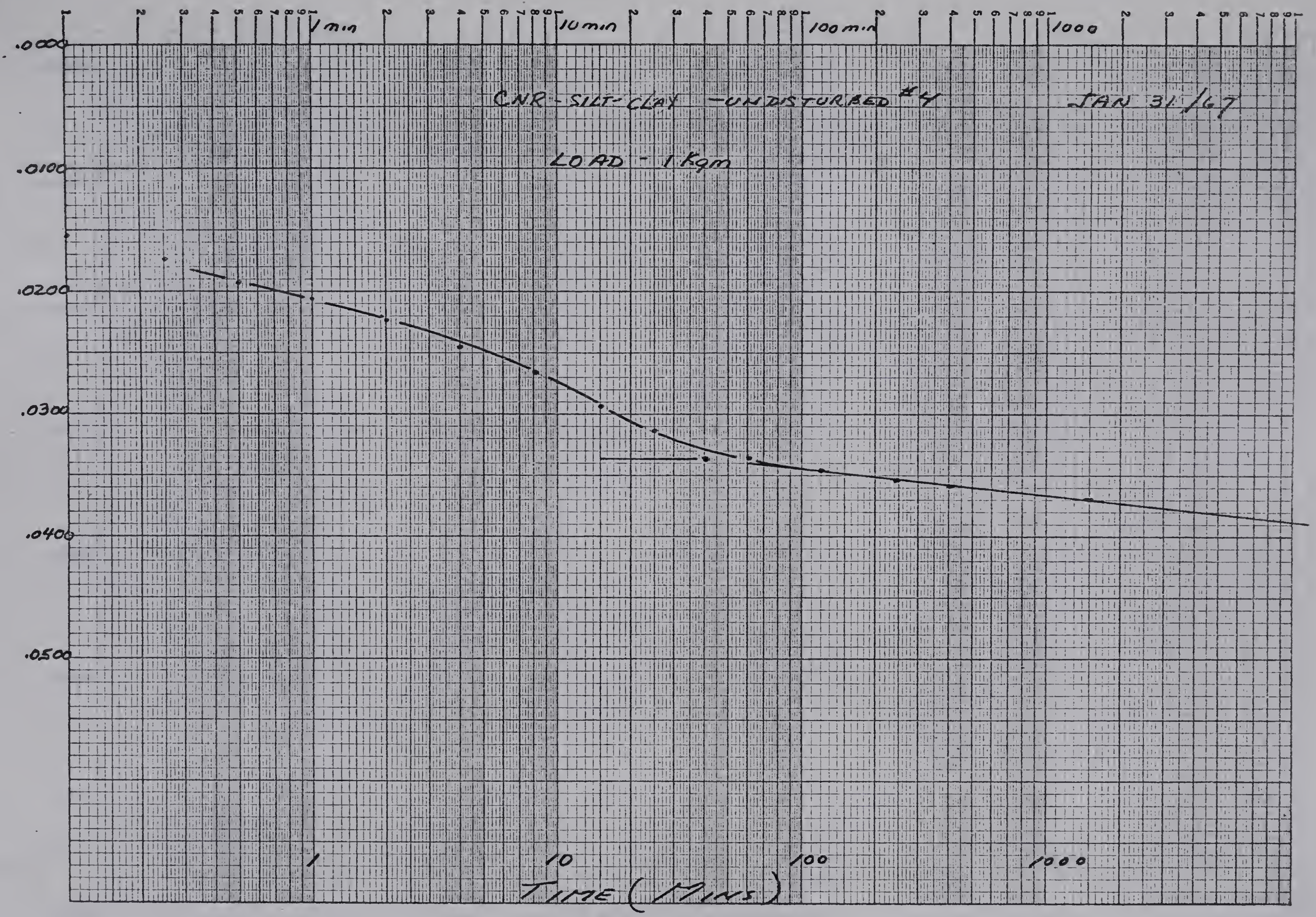
Specific Gravity of Soil Solids G_s = 2.73 Height of Soil Solids H_s = 0.333 ins.
Void Ratio e (End) = 1.29
Void Ratio e(Start) = 1.97
Void Ratio e (Start Dimensions) =

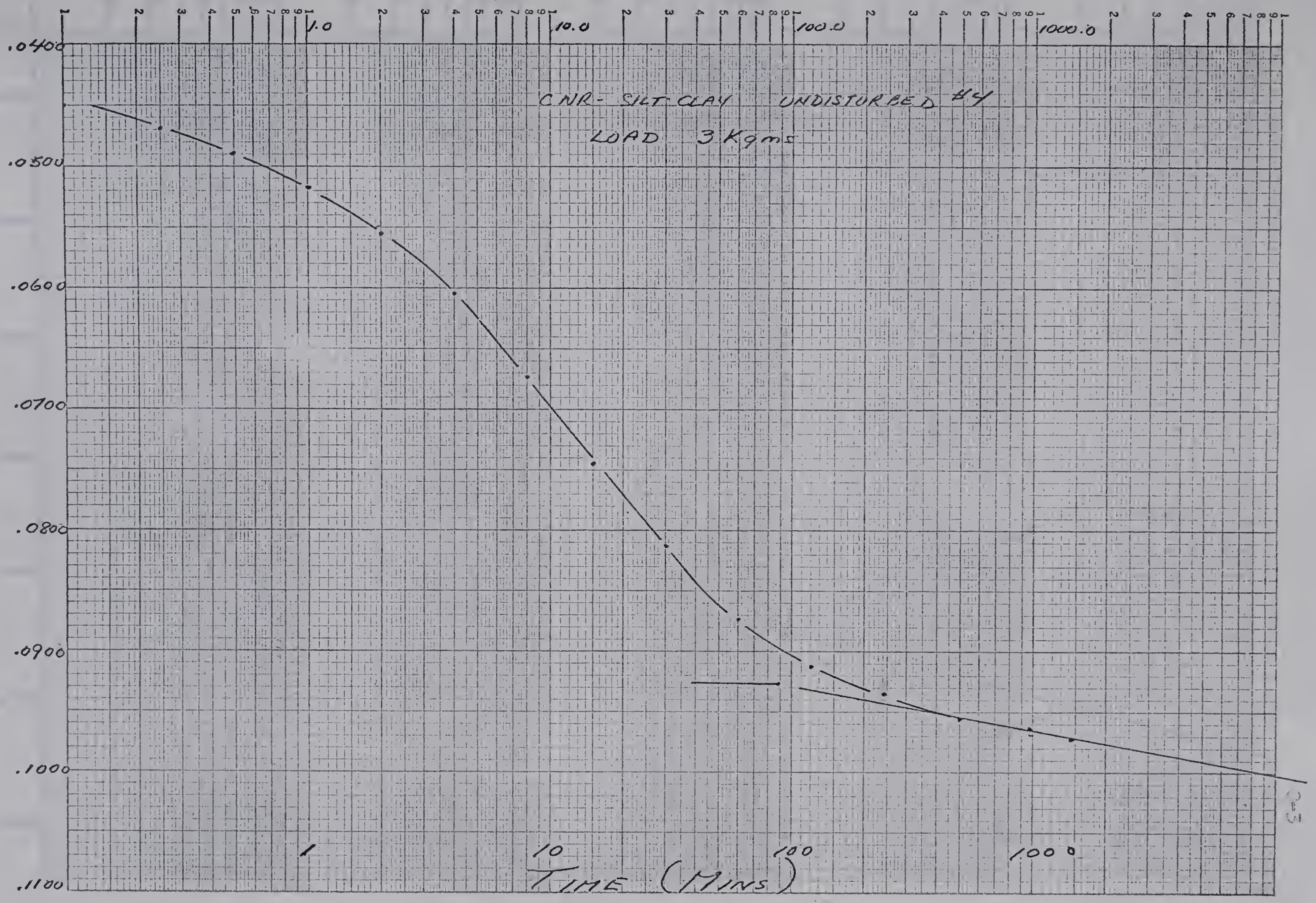
e(End) = W%(End) x G_s $H_s = \left(\frac{\text{Wt. Soil}}{G_s \times \text{Area} \times 2.54}\right)$ ins. e = previous e $\pm \frac{\text{Def'l.}}{H_s}$

Time Interval	Load on Pan (Kgms)	Corr. Dial Reading (ins.)	Deflection (ins.)	Deflection / H_s	Void Ratio 1.e 968	Pressure Kg/cm² = T/ft²
	0.3	.0088	.0012(s)	.004	1.972	.09
	1.0	.02670	.02686	.081	1.891	.33
	3	.08651	.05981	.180	1.711	1.00
	6	.14094	.05523	.166	1.545	2.00
	12	.20312	.06318	.190	1.355	4.00
	24	.26751	.06609	.199	1.156	8.00
	48	.32834	.06443	.194	.962	16.00
	12	.31599	.01775	.053	1.015	4.00
	3	.28700	.03179	.095	1.110	1.00
	0.3	.23797	.06983	.180	1.290	.09

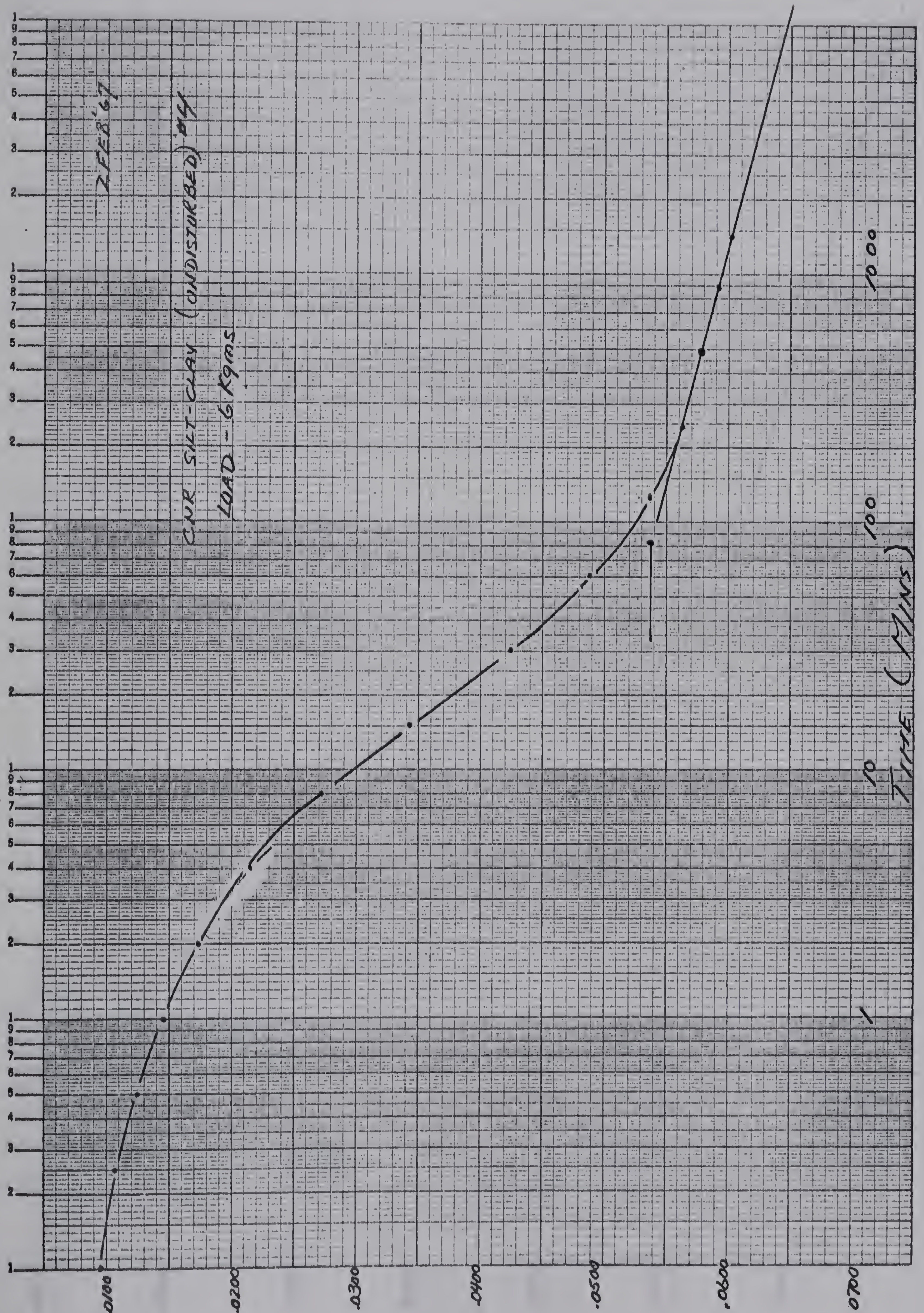
Compressive Index = 0.65
Swelling Pressure = ______ T/ft²
Pre-Cons. Load = ______ T/ft²

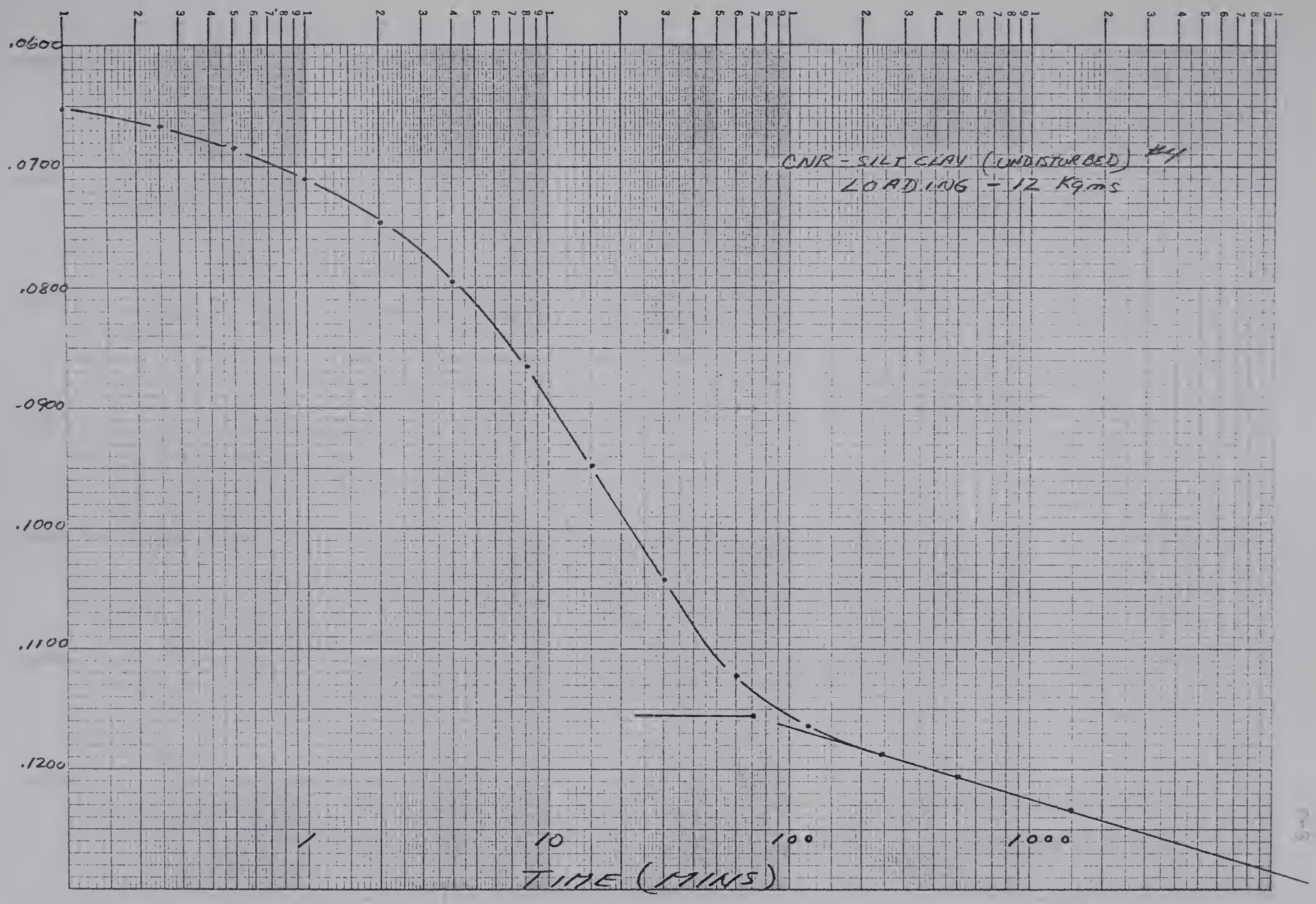


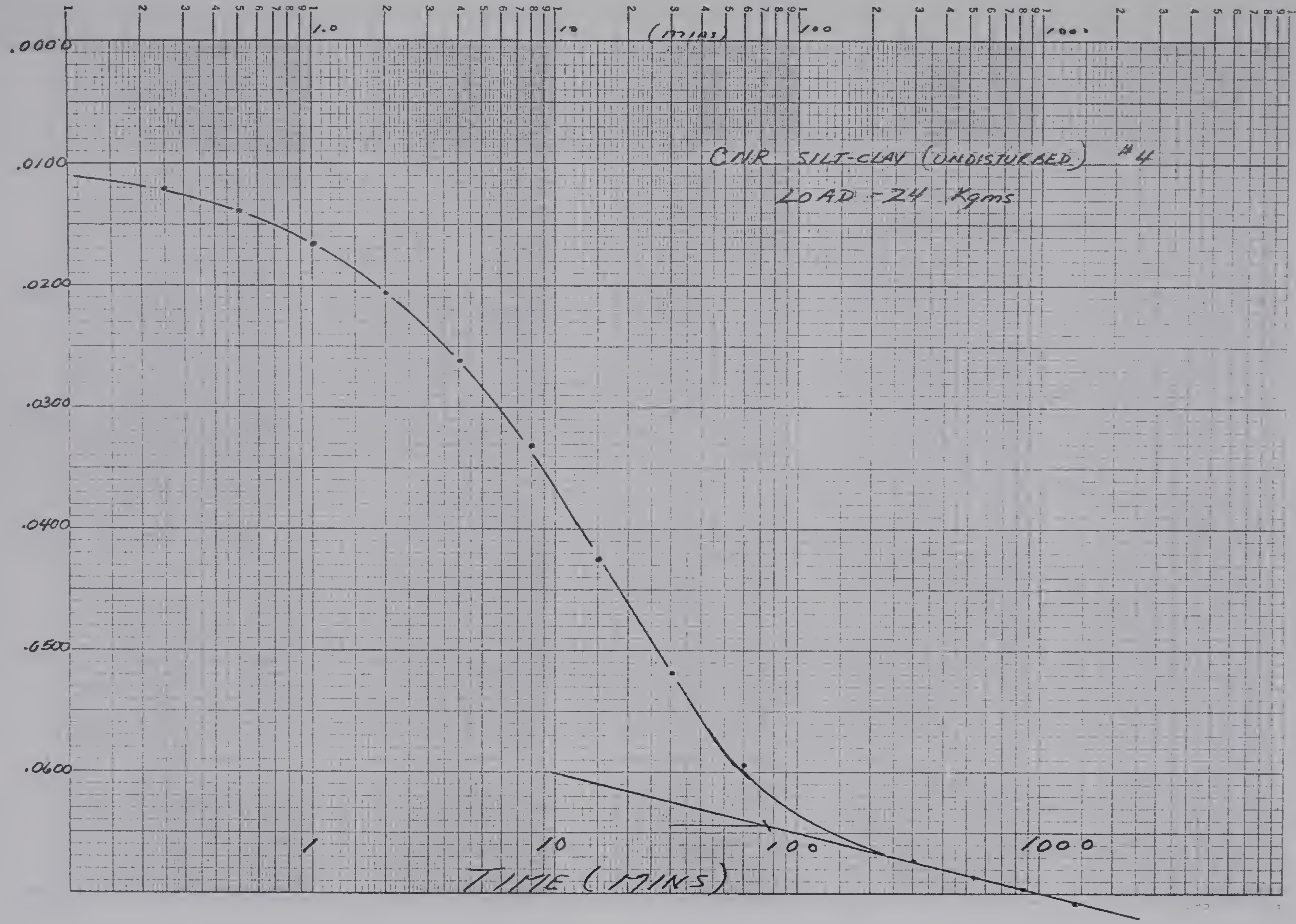

CNR - SILT - CLAY - UNDISTURBED #4
JAN 31/67
LOAD - 1 Kgm
1 min
10 min
100 min
1000
.0000
.0100
.0200
.0300
.0400
.0500
1
10
100
1000
TIME (MINS)



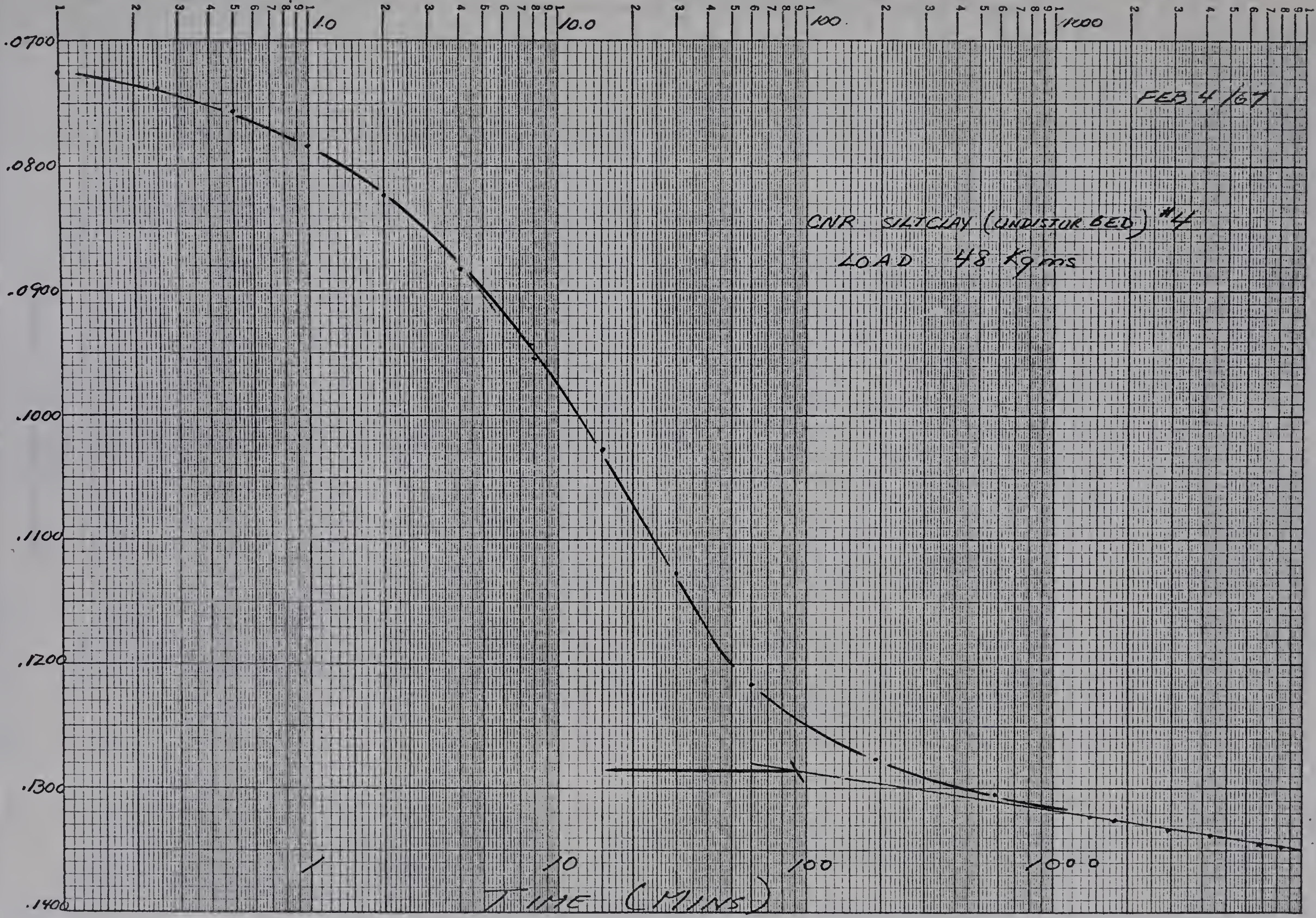
CNR- SILT-CLAY UNDISTURBED #4
LOAD 3 Kgms
.0400
.0500
.0600
.0700
.0800
.0900
.1000
.1100
1.0
10.0
100.0
1000.0
1
10
100
1000
TIME (MINS)


2 FEB '67
CNR SILT-CLAY (UNDISTURBED) MG
LOAD - 6 Kgms
TIME (MINS)
1
10
100
1000
.0100
.0200
.0300
.0400
.0500
.0600
.0700


CNR - SILT CLAY (UNDISTURBED) #4
LOADING - 12 Kgms
.0600
.0700
.0800
.0900
.1000
.1100
.1200
1
10
100
1000
TIME (MINS)



CNR SILT-CLAY (UNDISTURBED) #4
LOAD - 24 Kgms
(mins)
1.0
10
100
1000
.0000
.0100
.0200
.0300
.0400
.0500
.0600
1
10
100
1000
TIME (MINS)


FEB 4/67
CNR SILTCLAY (UNDISTURBED) #4
LOAD 48 Kgms
TIME (MINS)
1.0
10.0
100.
1000
1
10
100
1000
.0700
.0800
.0900
.1000
.1100
.1200
.1300
.1400

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
CONSOLIDATION RESULTS

PROJECT	S-101
SITE	DUNCAN LAKE
SAMPLE	110 7½"-11"
LOCATION	
HOLE	SH-6 DEPTH 179'-3 180'-11"
TECHNICIAN	WEJ DATE APR/67

Specific Gravity of Soil Solids G_s = 2.78 Height of Soil Solids H_s = .455 ins.

Void Ratio e (End) = .881

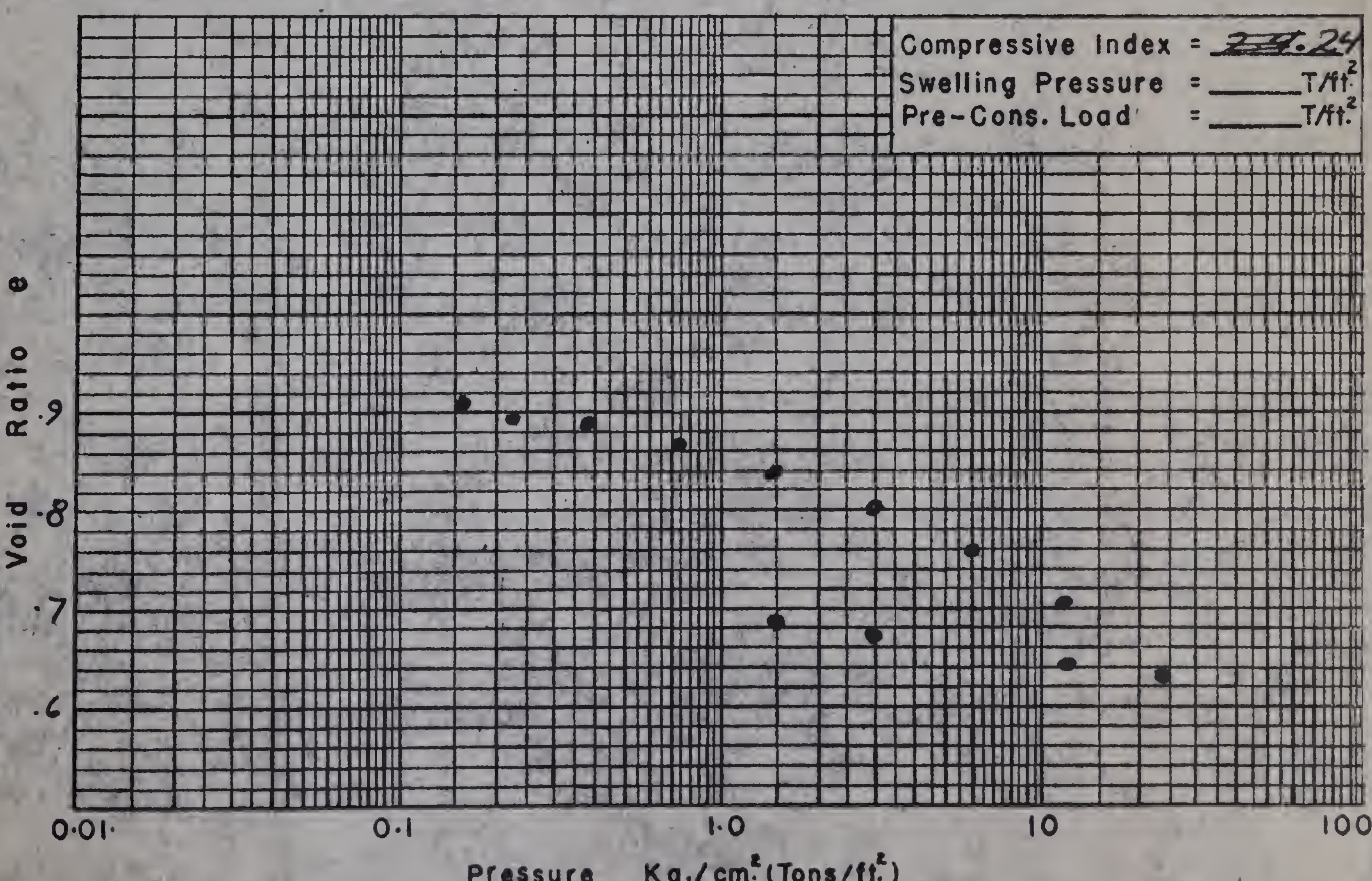
Void Ratio e(Start) = ~~.922~~ .800

Void Ratio e (Start Dimensions) = .922

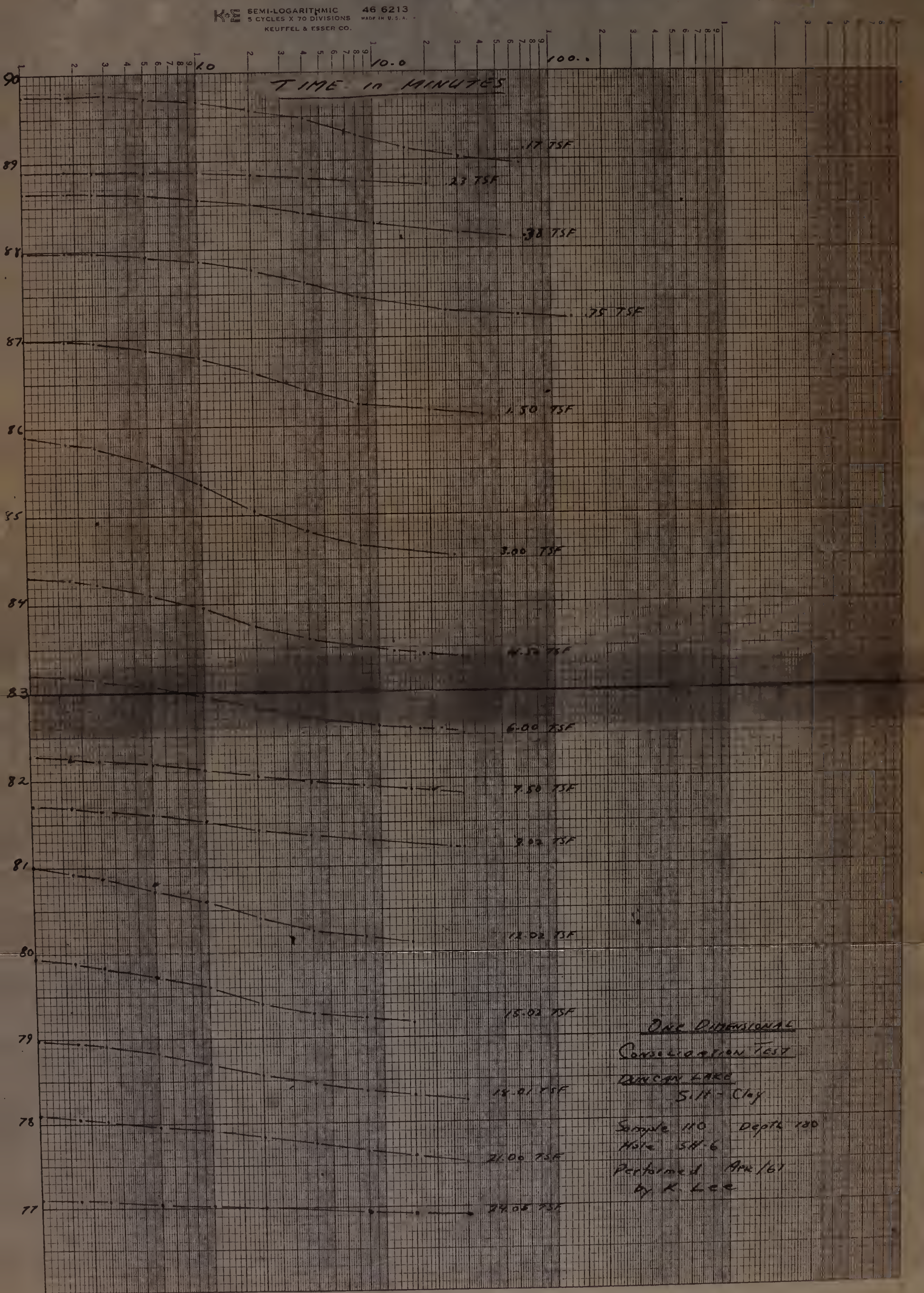
e(End) = W%(End) x G_s $H_s = \left(\frac{\text{Wt. Soil}}{G_s \times \text{Area} \times 2.54}\right)$ ins.   e = previous e $\pm \frac{\text{Def'l.}}{H_s}$

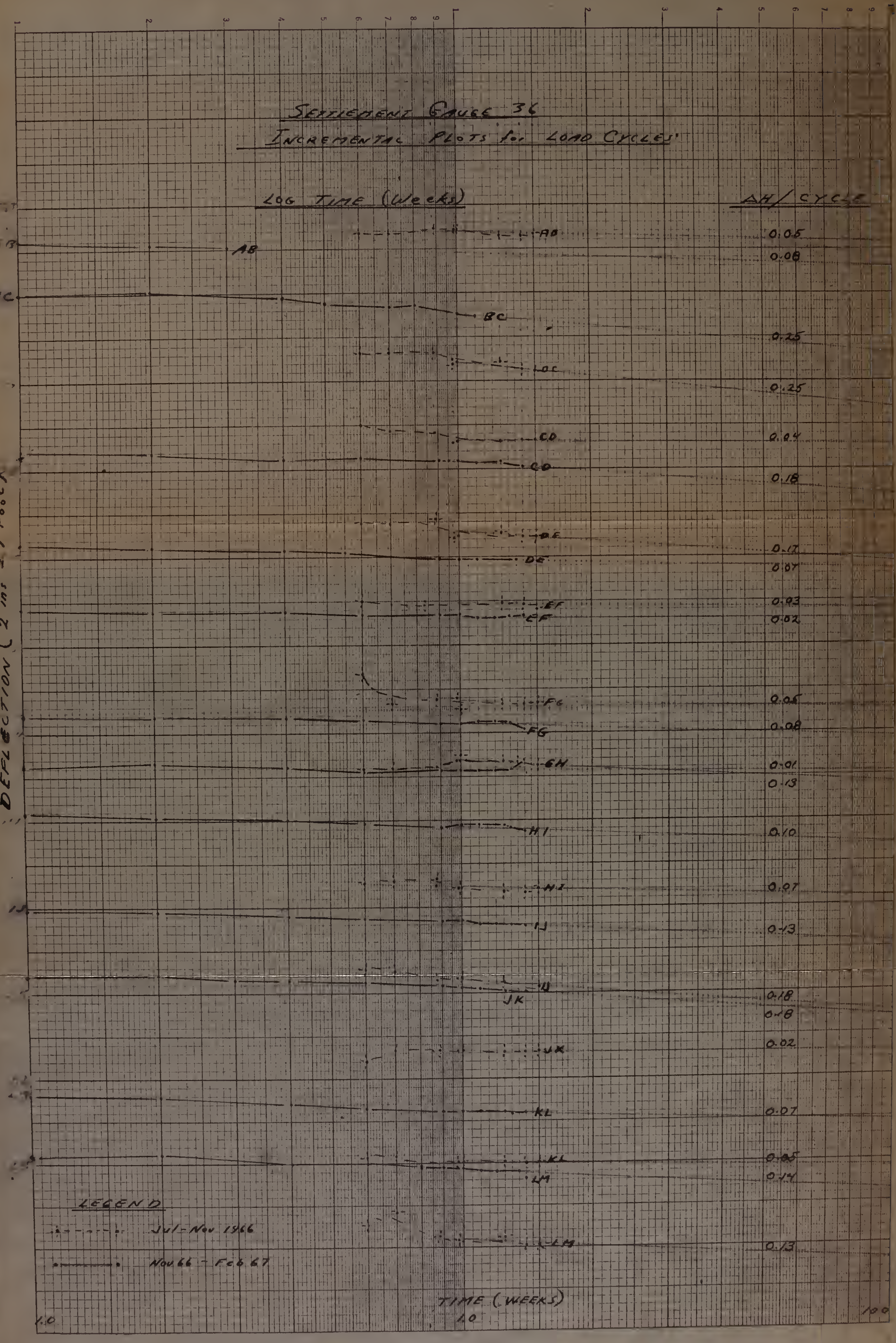
Time Interval	Load on Pan (gms)	Corr. Dial Reading (ins.)	Deflection (ins.)	Deflection / H_s	Void Ratio e	Pressure Kg/cm² = T/ft²
8 min		8930	.0070	.015	.902	0.17
8 "		8878	.0052	.011	.896	.23
8 "		8832	.0046	.010	.886	.38
8 "		8747	.0085	.019	.867	.75
8 "		8626	.0121	.027	.840	1.50
8 "		8464	.0162	.036	.804	3.0
8 "		8264	.0200	.044	.760	6.0
8 "		8020	.0244	.054	.706	12.0
8 "		7696	.0324	.071	.635	24.0
30 "		7690	.0006	.001	.634	24.0
8 "		7735	.0045	.010	.644	12.0
8 "		7857	.0122	.027	.671	3.0
8 "		7924	.0067	.015	.686	1.6
1080 "		8442	.0518	.113	.799	0

Compressive Index = ~~.23~~ .24
Swelling Pressure = ______ T/ft²
Pre-Cons. Load = ______ T/ft²



Copy of Test by:
Soil Mechanics Services
Vancouver


K+E SEMI-LOGARITHMIC 46 6213
5 CYCLES X 70 DIVISIONS MADE IN U.S.A.
KEUFFEL & ESSER CO.
TIME in MINUTES
1.0
10.0
100.
90
89
88
87
86
85
84
83
82
81
80
79
78
77
.17 TSF
.23 TSF
.38 TSF
.75 TSF
1.50 TSF
3.00 TSF
4.50 TSF
6.00 TSF
7.50 TSF
9.02 TSF
12.02 TSF
15.02 TSF
18.01 TSF
21.00 TSF
24.05 TSF
One Dimensional
Consolidation Test
Duncan Lake
Silt - Clay
Sample 110 Depth 180
Hole SH-6
Performed Apr/61
by K. Lee


K&E SEMI-LOGARITHMIC 2 CYCLES X 180 DIVISIONS 47 4972
KEUFFEL & ESSER CO.
SETTLEMENT GAUGE 36
INCREMENTAL PLOTS for LOAD CYCLES'
LOG TIME (Weeks)
ΔH/CYCLE
AB 0.05
AB 0.08
BC
BC 0.25
0.25
CD 0.04
CD 0.18
DE 0.17
DE 0.07
EF 0.03
EF 0.02
FG 0.05
FG 0.08
GH 0.01
0.13
HI 0.10
HI 0.07
IJ 0.13
JK 0.18
JK 0.18
JK 0.02
KL 0.07
KL 0.05
LM 0.14
LM 0.13
LEGEND
Jul-Nov 1966
Nov 66 - Feb 67
TIME (WEEKS)
DEFLECTION (2 ins = 1 Foot)
1.0
10
100